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LATERAL-PLATE AND RIGID-PILE TESTS IN BEACH SAND

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LATERAL-PLATE AND RIGID-PILE TESTS IN BEACH SAND

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by

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ABSTRACT

Seven lateral-plate bearing tests were performed in moist beach sand at six different depths from 1 to 9 feet. In addition, 25 rigid-pile tests were performed in the same soil with pile widths ranging from 1 to 16 inches and embedment depths from 12 to 66 inches. In conjunction with these experiments, determinations were made of the in situ vane shearing strength, density, moisture content, and standard penetration of the soil. The objective of these studies is to develop procedures for the determination of soil moduli including variations with depth, magnitude of deflection, and width of loaded area.

Among other information, the test results have shown that the coefficient of horizontal subgrade reaction, k_h, of the soil used decreases exponentially with increasing deflection and increases exponentially as depth increases. It was found that for different widths of loaded area, b, at some depth in this soil, k_h (b) is a constant for a constant value of deflection, y, divided by b. This indicates that k_h (b) should be expressed as a function of depth and y/b.

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INTRODUCTION

Subject and Purpose of Report

This report describes and presents the results of a series of lateral-plate tests and rigid-pile tests in moist beach sand. These experiments were performed under Task No. Y-F015-15-01-009, Lateral Plate Tests, sponsored by the U. S. Navy Bureau of Yards and Docks. The objective of this task is to develop procedures for the determination of soil moduli including variations with depth, width of loaded area, and magnitude of deflection. The specific objective of this study was to determine the value of soil modulus and its variation with magnitude of deflection, width of loaded area, and depth in a uniform moist granular soil deposit. Also, it was desired to determine if a relationship exists between soil modulus and a soil property which can be easily determined by an in situ soil test.

Analysis of the Problem

Design of structural systems to gain horizontal support from soil requires that the lateral load-deformation properties of the soil be known. These properties are difficult to determine accurately even for one specific soil deposit, because of their complex variations with rate of loading, size and shape of loaded area, magnitude of deflection, and overburden pressure.

A term used to represent the ratio of the change in horizontal soil pressure at a point on a structure to the corresponding horizontal deflection of that point is called a coefficient of horizontal subgrade reaction,[1] k_h and has units of psi per inch of displacement. Considerable information on subgrade coefficients is contained in Reference 1, where values of k_h are suggested for clays and granular soils based upon their consistency and relative density, respectively. This reference also describes the influence of size of loaded area on k_h and presents probable variations with depth for different soil types while assuming that the soil behaves elastically at stress levels of interest. The proposed values of k_h were intended for computation of stresses in earth-supported structures and were not meant for estimation of deflections. Consequently, reliable soil information to aid in the design of horizontally loaded soil-supported structures is lacking.

For large construction projects requiring piles to resist lateral loads, it may be economically justifiable to perform full-scale tests to determine the true capacity of the piles in advance of construction. However, for smaller jobs or for structures such as anchored bulkheads, buried conduits, and buried protective structures, full-scale tests are impractical and k_h must be determined by some other method before construction.

Two separate approaches have been considered. The first involves the development of an instrument to actually measure the in situ load-deformation characteristics, and the other is an attempt to correlate k_h with some easily determined soil property. The first approach has been taken by various researchers[2, 3] with some degree of success, and although the suggested soil tests are relatively difficult to perform, the techniques have considerable merit. A serious objection to this type of approach is the soil disturbance involved. In addition, the determination of k_h for designing buried structures requires a knowledge of the effect of deflection and overburden pressure both increasing simultaneously at a given depth, and this effect cannot be simulated economically in the field. Correlation of k_h with other soil properties seems to be the more desirable approach in that this method will allow a greater number of k_h determinations to be made at a given site and can be used economically on small construction projects. However, it is the more difficult to perfect and will require several full-scale tests for evaluation. This second method was attempted by a correlation of triaxial test results with the results of full-scale pile tests,[4] but the proposed technique seems to be impractical for small projects. In addition, the results are subject to sampling and testing errors. It would be more desirable to correlate k_h with soil properties determined by easily performed in situ soil tests such as vane shearing or penetration.

Background

Several theoretical analyses have been developed for soil-supported structures such as laterally loaded piles[5, 6, 7] and poles,[8, 9] buried conduits,[10, 11, 12] piers,[13] and protective structures.[14] The applicability of these theories is limited by a general lack of knowledge of the lateral load-deflection characteristics of the supporting soil. Summaries of the available test information on laterally loaded piles and poles are contained in References 6 and 9, respectively. Although several tests have been performed, the majority of the experimenters were concerned with one specific construction problem or one particular method of analysis; consequently, the reported results are often inadequate for treatment of other problems or for use in different analysis techniques. In addition, a large portion of the experimental results was reported without adequate information on soil properties, making it difficult to draw reliable conclusions. However, a search of the literature shows the general behavior which can be expected of these structures and serves to establish problem areas in testing and analysis.

Scope and Approach

This report is concerned with the determination of k_h and its variation with deflection, width of loaded area, and depth in artificially placed moist beach sand. The most severe one-time loading condition has been considered, whereby a load was applied and sustained until progressive deflection at that load level had ceased; repetitive loadings were not considered.

A test program was designed to determine the interrelation of the results of lateral-plate bearing tests at various depths, horizontal loadings of various diameters and lengths of rigid piles, and soil property determinations by both laboratory and in situ soil tests. The results of all the tests performed are reported herein with the results and conclusions of a data analysis.

A secondary objective of this test program was to determine the most efficient course for future research. A later section of this report describes the anticipated future testing based upon the results of this experimental program. The Appendix contains a brief analysis of a laterally loaded rigid pile in a layered soil system presented to show the influence of soil modulus, and its variation with depth, on the response of a laterally loaded rigid pile.

TEST SETUP

Figure 1 shows a typical setup for the lateral-plate and rigid-pile tests. At the time this photograph was taken, the plate device was in position for a test with the centerline of the plate at a depth of 60 inches. It was covered with a tarpaulin to protect the instrumentation from sand and moisture. The test items shown are described in more detail in later sections of this report.

Soil Placement

The procedure for placement of the lateral-plate apparatus and the piles was generally the same for all tests. An area approximately 25 feet square was excavated to a depth just above the water table at its highest point after high tide. The plate-test apparatus was then guyed upright at the proper position while sand was being placed in shallow layers by a crane with a "clam" bucket. As the sand was being placed, the area was constantly being soaked with water from a fire hose to effect uniform compaction. When the desired pile tip elevations were reached, the piles were placed upright and held in position until sufficient soil was placed to make them stable. The compaction procedure was continued until the final desired surface elevation was reached, after which soaking was continued for approximately 1 more hour to saturate the area uniformly. As soon thereafter as possible, the various tests were performed, and the in situ soil properties were determined. All test objects were then removed and the procedure was repeated for the next plate depth in the test series.

Soil Property Determinations

During each test it was necessary to determine such soil properties as moisture content, density, and in situ vane shearing strength, using procedures which were similar to those widely used. In addition, the standard penetration test was performed, resulting in values of N defined as the number of blows required of a 140-pound weight falling 30 inches to drive a 2-inch-OD by 1-3/8-inch-ID split-spoon sampler a depth of 1 foot. Densities were determined by the sand-replacement method, using precalibrated Ottawa sand. Samples for determining moisture content were obtained from soil specimens taken during the standard penetration tests and the density measurements.

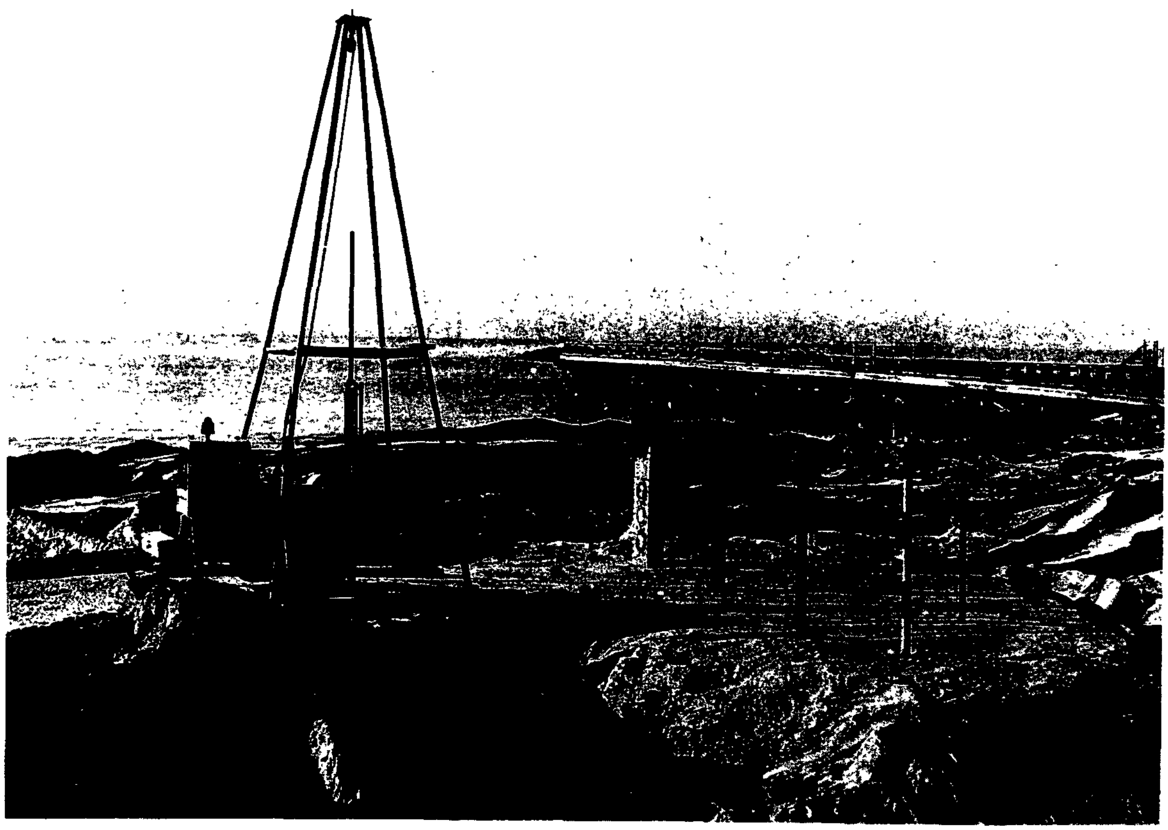


Figure 1. Drilling rig for determining soil properties; plate apparatus; and piles with diameters of 6, 4, 1, 3, and 2 inches.

The vane shearing device used had a width of 2.5 inches and a height of 5 inches and was made up of four stainless-steel vanes welded to a 3/4-inch-diameter shaft. After it was driven to some depth in the soil with a sledge hammer, the maximum torque required to turn the device at that depth was measured by a torque wrench with a maximum capacity of 600 inch-pounds. Since this test, as performed, required very little time, it was possible to make several measurements and thereby obtain reliable average values of the vane shearing strength for each test setup.

Friction on the turning shaft was measured by removing the vanes and determining the torque necessary to turn the shaft alone. In all cases the friction correction was a small percentage of the shearing strength of the soil.

Results of the vane shearing strength tests are discussed later. A mechanical analysis of the beach sand is shown in Figure 2. Average values of the remainder of the soil properties are tabulated below.

Dry density, γ_d	100 lb/ft^3
Moisture content, w	6 percent
Relative density, D_r	65 percent
Specific gravity, G	2.63
Standard penetration, N	2-4 blows/ft

For the minimum density determinations, the sand was placed in a 1/30-cubic-foot steel cylindrical mold in 1-inch layers with a funnel having a 1/2-inch-diameter spout. Sand was allowed to flow slowly from the funnel by keeping the spout in contact with the bottom of the mold or the surface of the previously placed layer while moving the funnel in a slow spiral motion working from the side of the mold toward the center. To determine the maximum density, the sand was placed in the same manner as during the minimum density determinations, except that each 1-inch layer was compacted thoroughly by a light tamping action using a hand tamper consisting of a 1-15/16-inch-diameter footing weighing 2.6 pounds. After the mold was filled, a concrete vibrator was attached to its side, and the sample was vibrated for 1 minute.

Instrumentation

Instrumentation was provided to measure the horizontal load on and resulting deflection of the rigid piles and the 9.75-inch-diameter plate. Response of the electronic instrumentation (to be explained later) was measured by Baldwin Type M strain indicators.

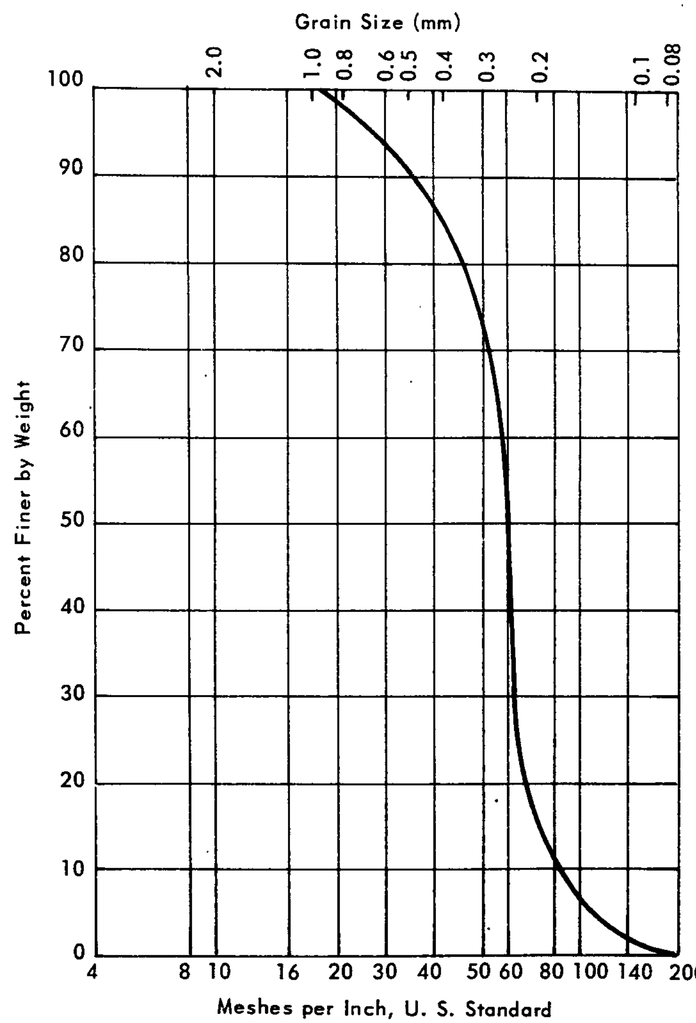


Figure 2. Grain-size distribution of beach sand.

Lateral-Plate Tests. Deflection of the plate was measured by Bourns linear-motion potentiometers, Model 108, with a travel capacity of 1.31 inches. Three of these transducers were mounted as shown in Figure 3 and interconnected electronically so that the average deflection of the three transducers could be noted by the response of one strain indicator. This system was calibrated by relating the response of the strain indicator with the deflection shown by an Ames deflection dial mounted to the center of the plate as the plate was deflected through its 1/2-inch capacity.

The load on the plate was measured by a load bar, also shown in Figure 3, with an SR-4 strain gage (Baldwin Type AB-7) mounted on the tension face. This load bar was calibrated in a Baldwin testing machine to a maximum load of 6000 pounds.

During the plate tests, two dial indicators (0.001-inch divisions) were mounted to the back of the pile to determine if any movement of the apparatus occurred from movement of the plate.

Rigid-Pile Tests. Deflections of the piles were measured by four Ames deflection dials, mounted as shown in Figures 4 and 5, in order that both the slope and the deflection of the pile at the ground surface could be determined. These dial indicators had a travel capacity of 2 inches and could be read accurately to the nearest 0.001 inch.

Horizontal loads on the piles were measured by a Dillon dynamometer placed in series with the hydraulic ram and the 1-inch-diameter cable. Depending upon the expected load-carrying capacity of the particular pile tested, either a 1-kip or a 10-kip-capacity dynamometer was used. They were both calibrated in a universal testing machine before they were used in the tests.

Testing Sequence

Seven lateral-plate tests and 25 rigid-pile tests have been performed; these tests are listed in chronological order in Table I. The following discussion describes these tests, the procedures for which were held constant during the program so that all results would be comparable.

LATERAL-PLATE TESTS

Description

The apparatus used to perform the lateral-plate tests consisted of a steel 16WF36 beam with 3/8-inch-thick cover plates bolted to the outer edges of the flanges to form a rectangular pile with exterior dimensions 16 inches by 7 inches. A movable circular plate 9.75 inches in diameter was mounted flush with the 16-inch side of the pile at a point 30 inches from the pile tip. Mounted to the web of the wide-flange beam were a geared driving mechanism for applying loads to the plate and the associated equipment necessary to measure the load on the deflection of the plate. These components can be seen in Figure 3. Before backfilling the apparatus for a test, a thin plastic film was taped over the plate to waterproof the electronic equipment.

After installation of the assembly to some depth below the ground surface, the plate could be forced laterally through its full 1/2-inch travel capacity by turning a wheel at the top of the pile as shown in Figure 6. Simultaneously, the applied load and plate deflection were registered by the previously described instrumentation.

Test Program and Procedure

The lateral-plate bearing test was the first test to be performed after setting up an experiment. A small increment of load (~ 0.5 psi) was slowly applied to the plate, and the corresponding deflection was recorded. The load was maintained constant, and deflection readings were taken every 5 minutes until the additional deflection during a 5-minute interval was less than 0.001 inch. At this time, the next load increment was applied, and the procedure was repeated. A test was usually discontinued after a deflection of 0.4 inch, at which time the load on the plate and its corresponding deflection was decreased until the load became equal to that of the at-rest pressures before the test. This was done to prepare the system for a subsequent rigid-pile test which will be described later.

During each plate test, two Ames deflection dials mounted to the back side of the pile were monitored to provide for corrections, if necessary, to the deflection of the plate.

Test Data

The results of the seven lateral-plate bearing tests are presented on Figure 7 in the form of curves relating plate deflection to the applied load after the deflection had stabilized at a given load level. The actual load-deflection curves had a stair-step configuration showing the progressive deflection at each load level. Since the major interest lies in determining the maximum deflection which will result from a given load, only the maximum values are considered in the following analysis.

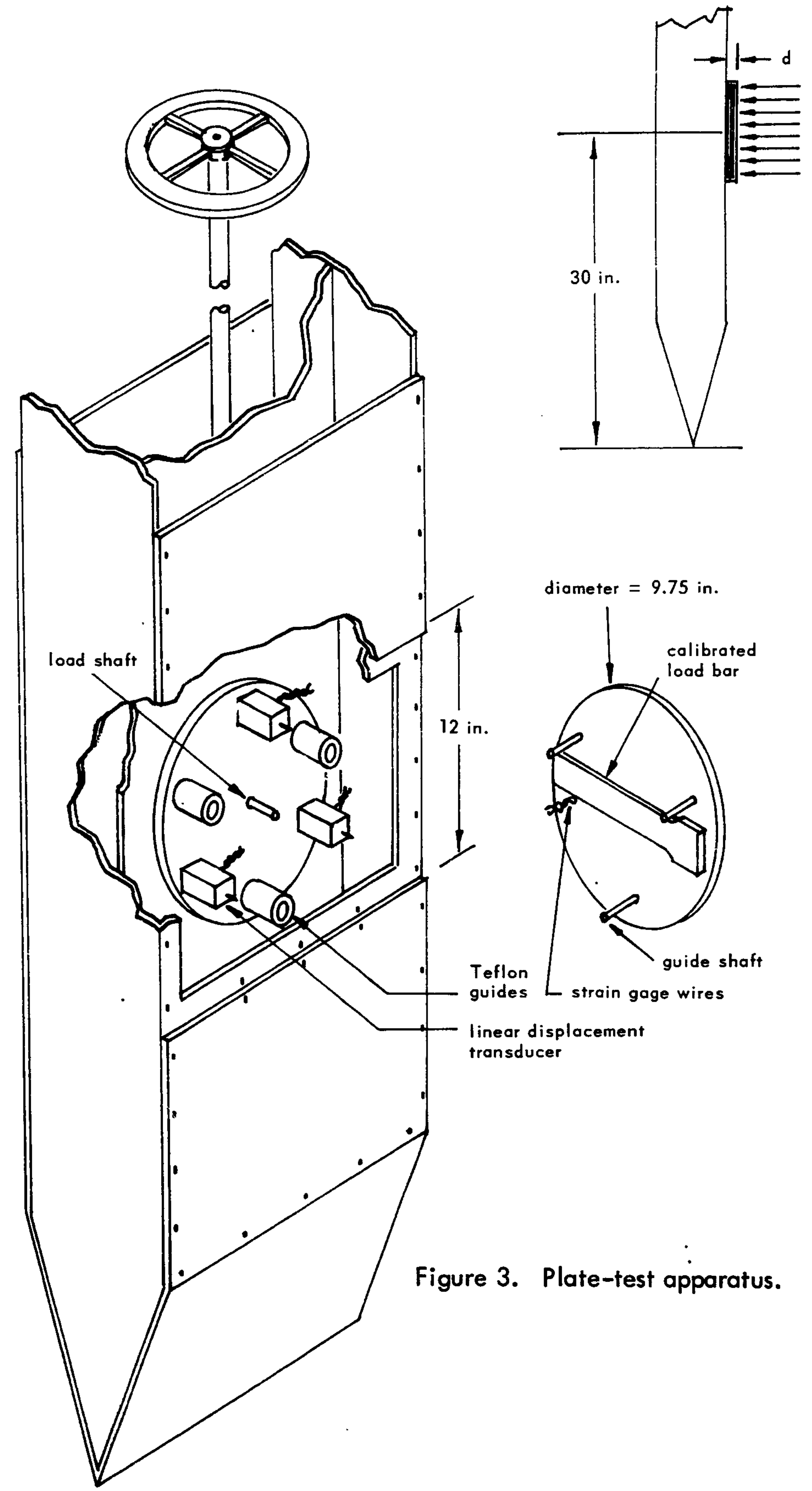


Figure 3. Plate-test apparatus.

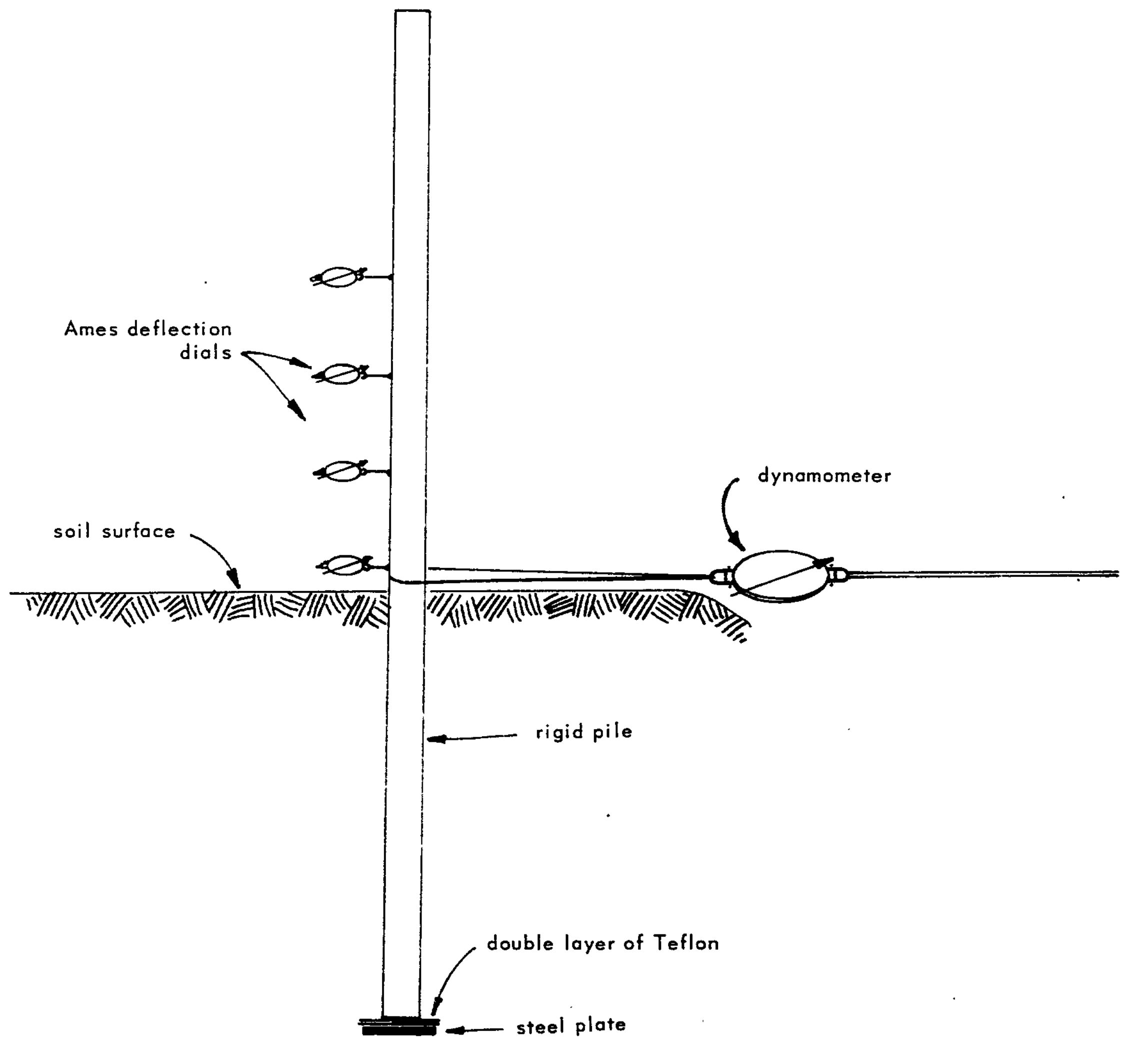


Figure 4. Pole test setup.

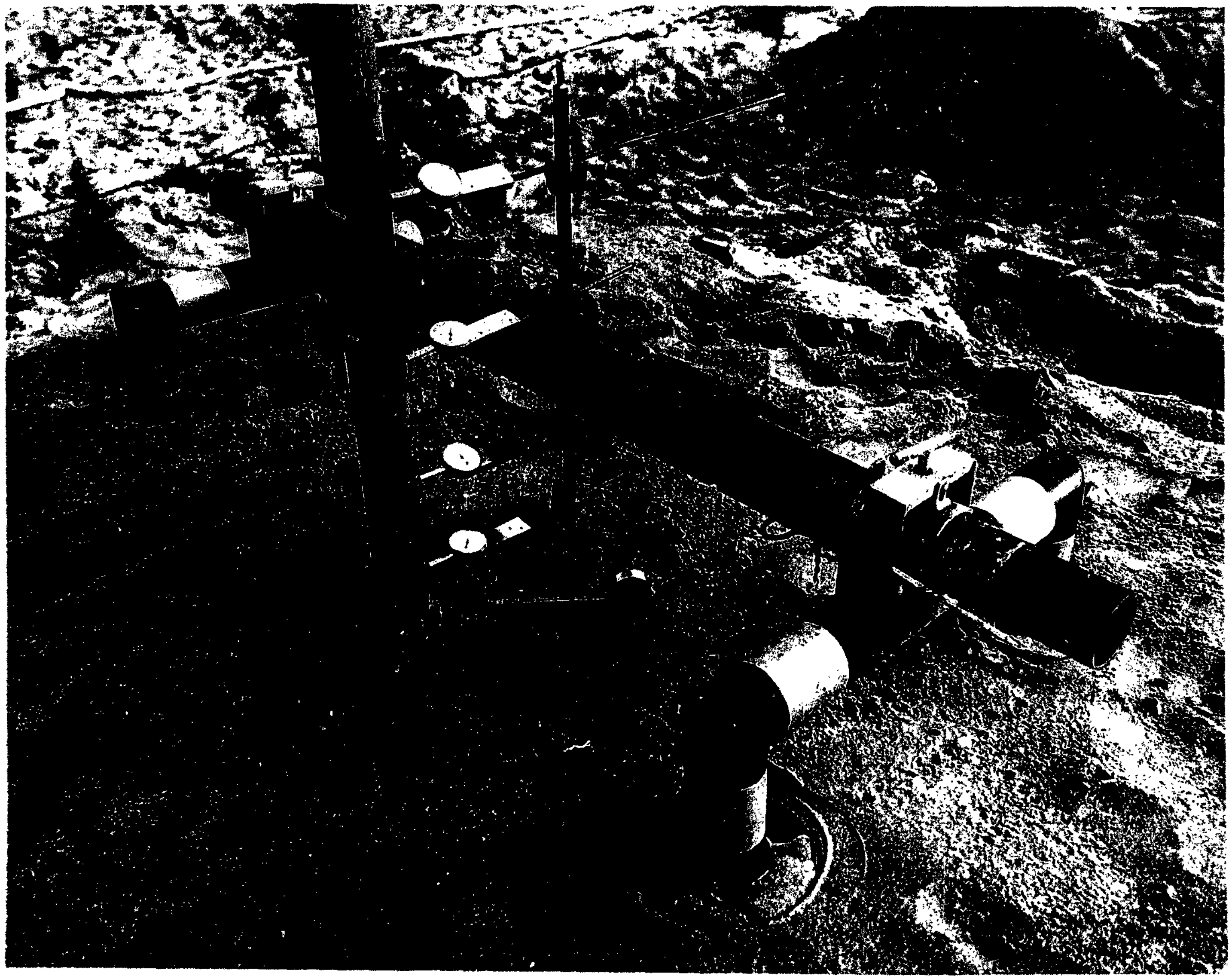


Figure 5. Pole test, showing positioning of Ames deflection dials.

Table I. Test Program

Date	Pile Test No.	Embedded Length, L (in.)	Pile Width, b (in.)	Plate Test No.	Depth to Centerline, X (in.)
10- 1-63	RP-1	18	2		
10- 2-63	RP-2	18	3	PL-1	12
10- 3-63	RP-3	36	4		
	RP-4	36	6		
	RP-5	42	16		
10-10-63	RP-11	66	16	PL-2	36
10-11-63	RP-6	12	1		
	RP-7	17	2		
	RP-8	30	3		
10-14-63	RP-9	34.5	4		
	RP-10	48	6		
10-17-63	RP-12	15	1	PL-3	60
	RP-13	18	2		
10-18-63	RP-14	36	3		
	RP-15	36	4		
	RP-16	48	6		
	RP-17	66	16		
10-22-63	RP-18	48	6	PL-4	84
10-29-63	RP-19	30	4	PL-5	108
10-30-63	RP-25	54	16	PL-6	24
10-31-63	RP-20	15	1		
	RP-21	15	2		
	RP-22	30	3		
11- 1-63	RP-23	36	4	PL-7	24
	RP-24	36	6		

Figure 6. Plate test.

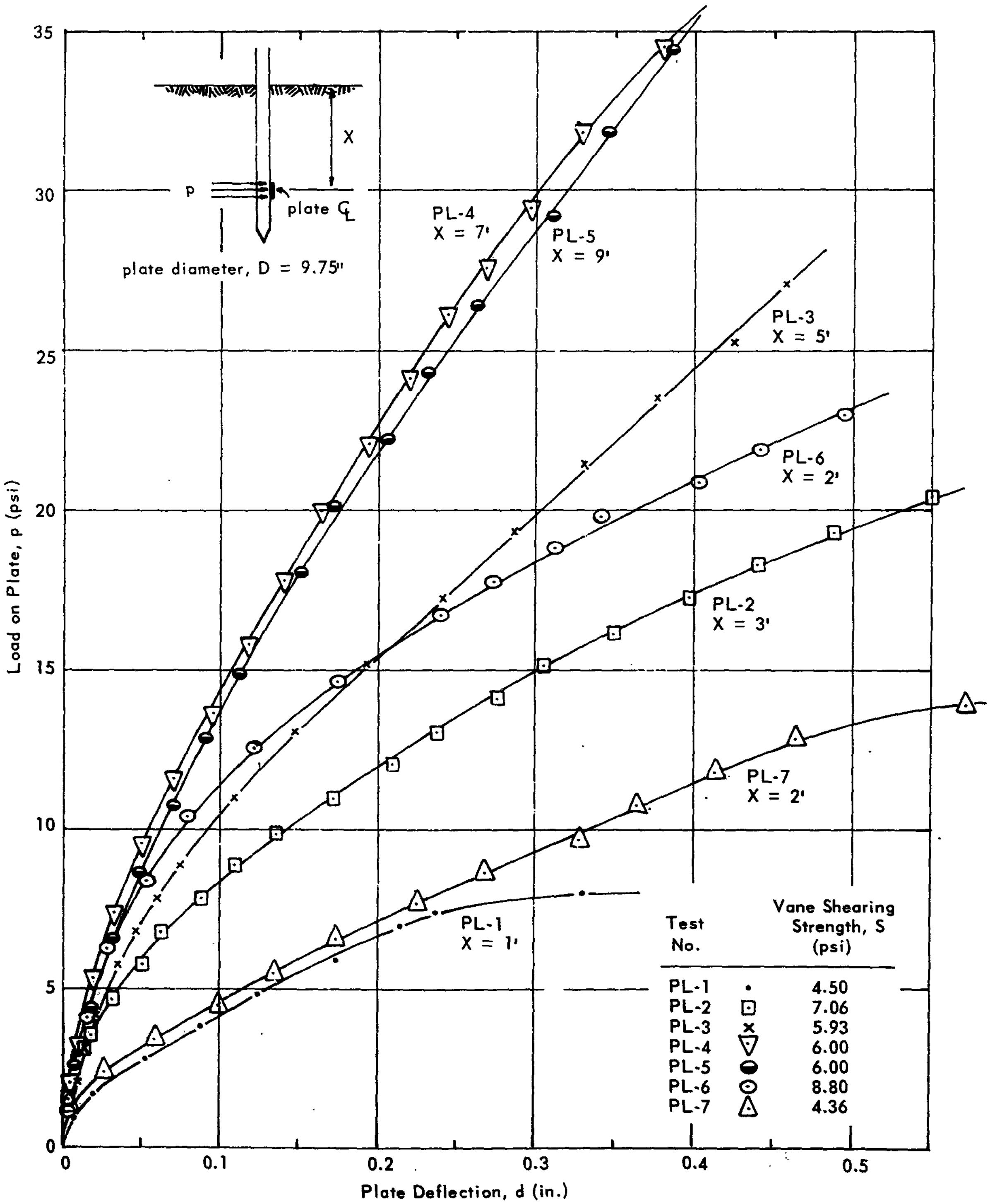


Figure 7. Lateral-plate bearing test data.

The data plotted in Figure 7 indicate that there is a general increase in soil stiffness and lateral bearing capacity with depth, as one would expect in a uniform granular soil deposit. However, the erratic relative positioning of the curves for tests PL-4, PL-5, and PL-6 indicates that soil conditions were not the same for all tests. This was substantiated by the vane shearing data plotted in Figures 8 through 10. A study of these soil data showed that the positions of the plate-test curves depended to some extent upon the shearing strength of the soil at the plate elevation during the respective tests. On this basis, an attempt was made to correct the data for these soil irregularities in order that comparable test results could be obtained.

To simplify correction of the data, it was assumed that the load on the plate necessary for a given deflection was directly proportional to the shearing strength of the soil in the vicinity of the plate. The validity of this assumption is substantiated by the general knowledge that the stiffness of soil increases as its strength increases.

The next step in the data-correction process was to determine a typical variation of shearing strength with depth upon which to base the corrections. This typical variation is shown in Figure 11 and is seen to be approximately an average of all shearing-strength data taken during the test program. Each curve in Figure 7 was multiplied by a factor obtained by dividing the normalized value of shearing strength at the depth under consideration (from Figure 11) by the actual value which existed during the corresponding test, as tabulated in Figure 7. The correction factors so obtained are tabulated in Figure 12.

A plot of the corrected lateral-plate test data is shown in Figure 12. By use of the aforementioned corrections, a set of load-deflection curves has been produced which follow a logical pattern, with the exception of the curve for test PL-7. A check of the field data revealed that the unusual shape of the curve for test PL-7 may have been due to a slight difference in test preparation procedures. Whereas the test apparatus had been removed from the ground before the other tests, test PL-7 was to be a repeat of test PL-6, and only the soil on one side of the apparatus was removed and recompacted. Evidently, this technique was not satisfactory, but it should be noted that although soil strength and compaction procedures for test PL-7 were quite different from those for test PL-6, their corrected load-deflection curves are very similar. The results of test PL-7 are not considered in the following data analysis.

Data Analysis

An inspection of the test data showed that although the test conditions represented were highly idealized, in that soil conditions, rate of loading, and plate size were made constant, the load-deflection relationship of the plate was dependent upon both plate depth and lateral deflection. Utilization of the test data to help predict the behavior of laterally loaded structures required the determination of a representative soil modulus and, if possible, a means for predicting its variation with depth and lateral deflection. The influence of different soil conditions, rates of loading, sizes of loaded area, and repetitive loading must await further research.

At a given magnitude of plate deflection, the corresponding values of plate load were obtained from Figure 12 and divided by that deflection to obtain the variation of secant modulus (coefficient) of horizontal subgrade reaction, k_h, with depth. The results of these computations, plotted in Figure 13, show that the variation can be approximated by two straight lines, one of which begins at the origin and intersects the other at a depth of approximately 2 feet. The slope of the first line, which corresponds to m_h as defined in Reference 1, decreases with increasing plate deflection, while the slope of the second line varies only slightly with deflection. Therefore, no value of m_h can be obtained to define accurately the variation of k_h with depth in the type of soil used during this test program.

It should be noted that there is a similarity between the variation of k_h with depth shown in Figure 13 and the typical variation of shearing strength with depth shown in Figure 11. This type of variation with depth is probably brought about by apparent cohesion in the sand. At shallow depths (less than 1 inch), the sand dries out and loses its cohesiveness. From the surface to a depth of about 2 feet, it appears that the additional shearing strength gained from both apparent cohesion and increased overburden pressure increases with depth. Below a depth of about 2 feet, the increase in shearing strength of the sand due to apparent cohesion is constant with depth; the increase in k_h below that depth is due mainly to the increase in overburden pressure with depth.

Since the plots in Figure 13 were each based upon only six data points, it was thought possible that the two straight lines could be represented approximately by an exponential variation of k_h with depth. A log-log plot, Figure 14, was made of the same information shown in Figure 13. It was found that the data for plate deflections greater than 0.10 inch (a deflection equivalent to about 1 percent of the plate width) could be reasonably represented by a series of straight lines on the log-log plot. The equations of the lines drawn in this manner took the form of

$$k_h = I_k X^{S_k} \tag{1}$$

where k_h is the secant modulus of horizontal subgrade reaction of the soil in lb/in.^{3}, X represents the depth to the plate centerline in inches, I_k denotes the intercept or the value of k_h for the line under consideration when X equals 1 inch, and S_k denotes the slope of the respective line. This equation can easily be evaluated for any of the lines shown on Figure 14. However, as previously noted, k_h is a function of both depth, X, and deflection, d, even for the simplified case under consideration. Therefore, the parameters I_k and S_k must have some relationship to the plate deflection, d. A log-log plot of this information, taken from Figure 14, revealed that I_k and S_k both could be expressed as separate functions of deflection, of the form

$$I_k = i_i d^{s_i} \tag{2}$$

$$S_k = i_s d^{s_s} \tag{3}$$

where the i and s terms here have meanings similar to the I_k and S_k terms in Equation 1. The best representative values of the i and s terms were determined and substituted into Equations 2 and 3, which were in turn substituted into Equation 1 to obtain

$$k_h = 1.66d^{-1.103} X^{0.86d^{0.330}} \tag{4}$$

Since k_h at any deflection is equal to the corresponding plate load divided by that deflection, the equation for plate load, p, in units of psi can be written in the form

$$p = 1.66d^{-0.103} X^{0.86d^{0.330}} \tag{5}$$

to represent the data plotted in Figure 14.

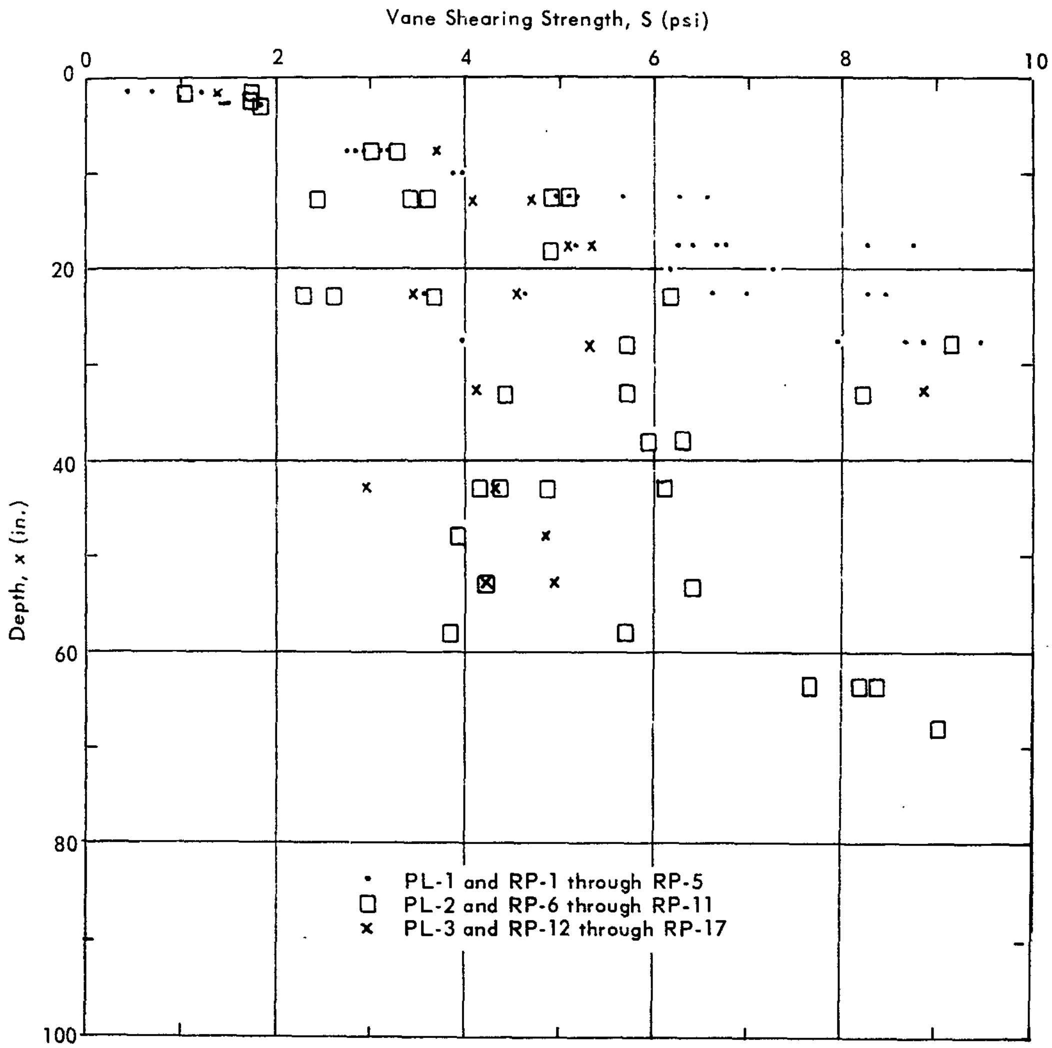


Figure 8. Vane shearing strength data.

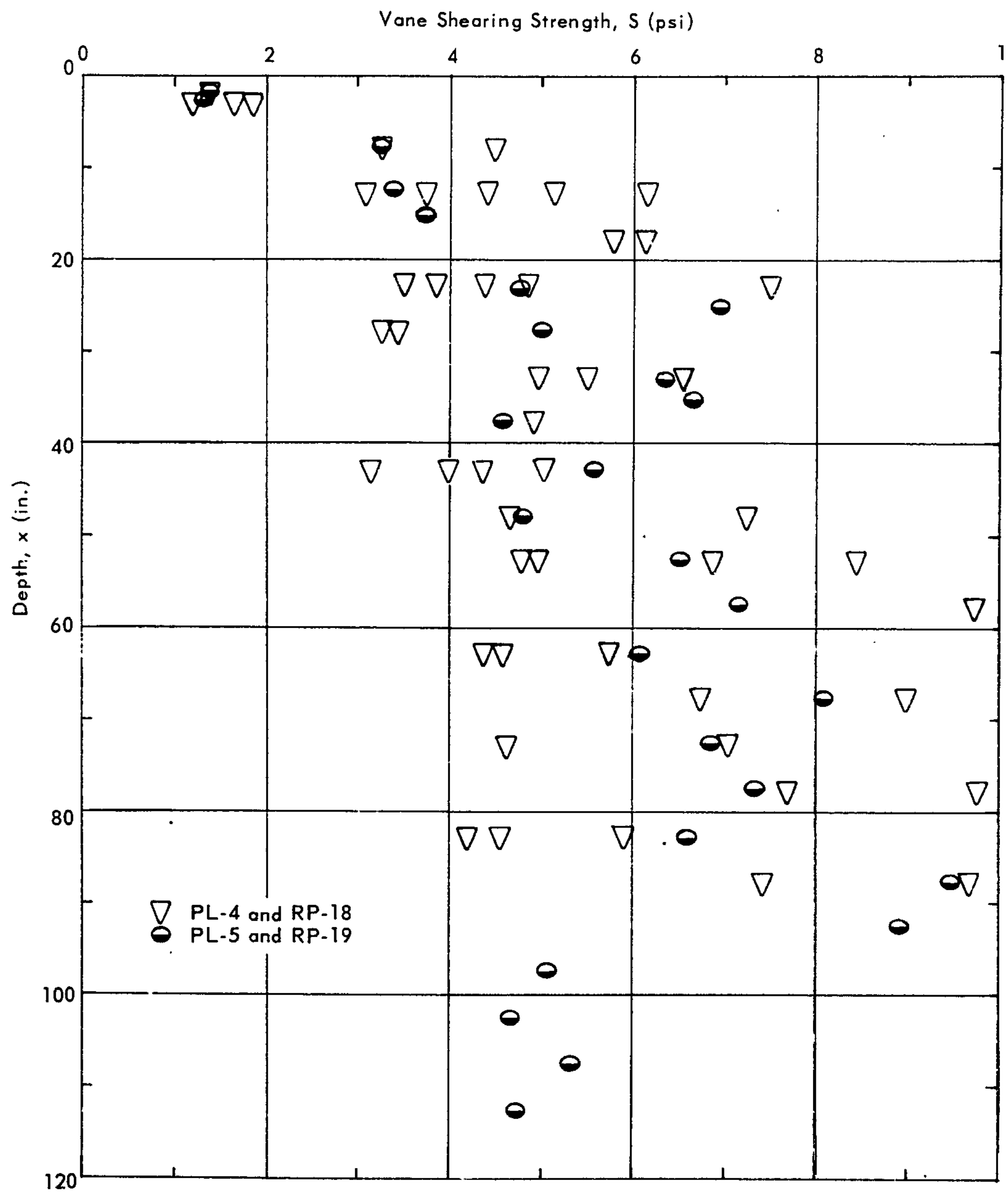


Figure 9. Vane shearing strength data.

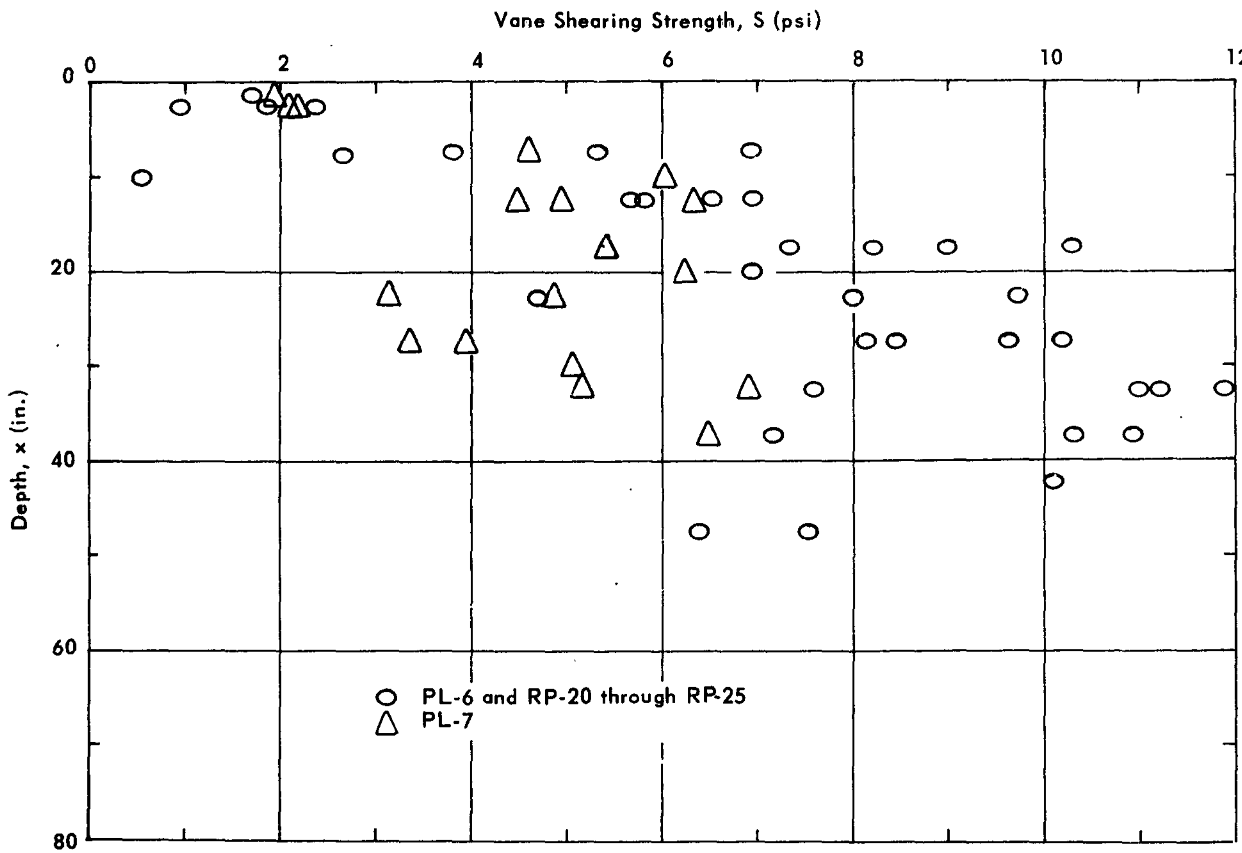


Figure 10. Vane shearing strength data.

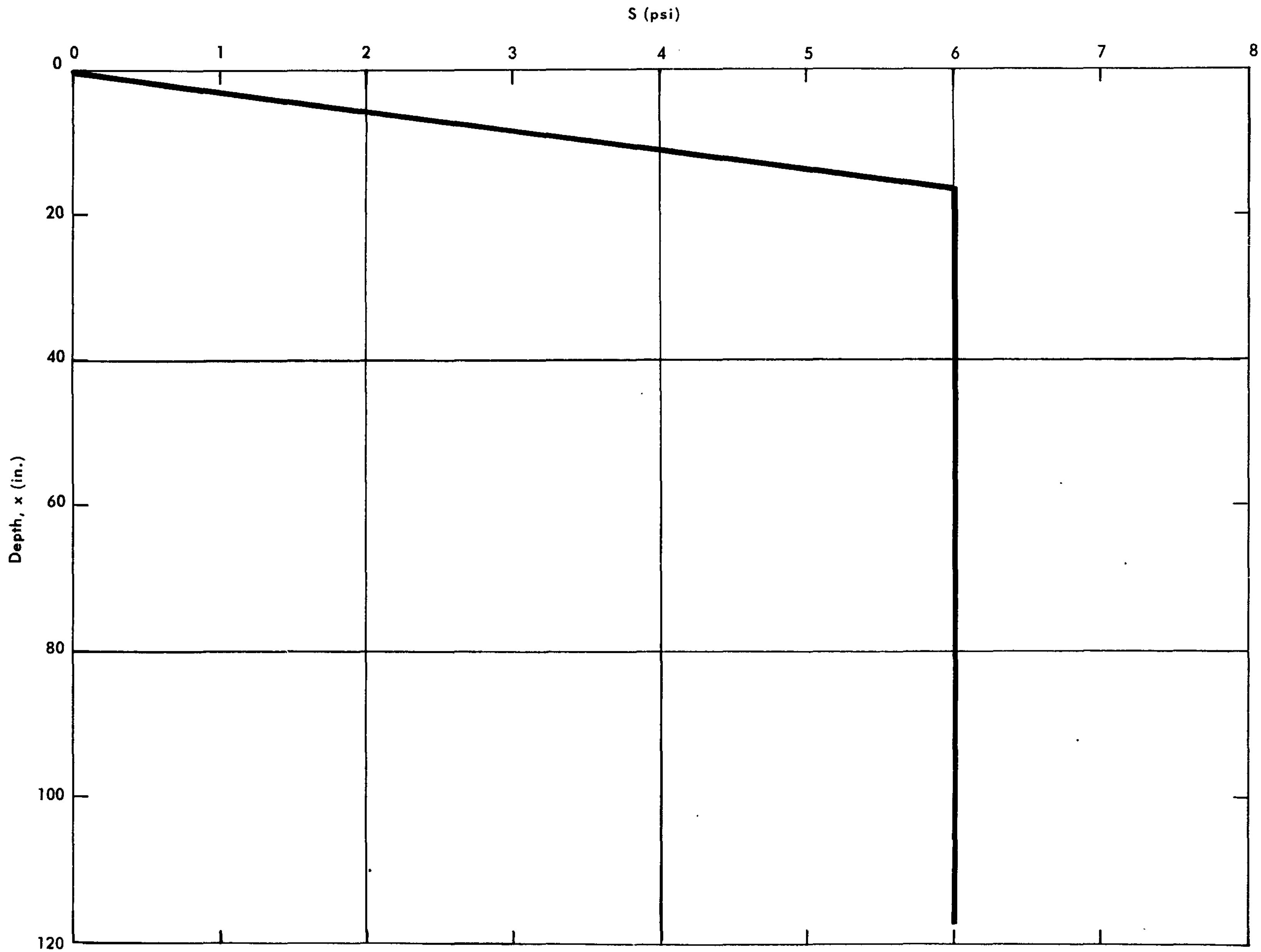


Figure 11. Average vane shearing strength versus depth.

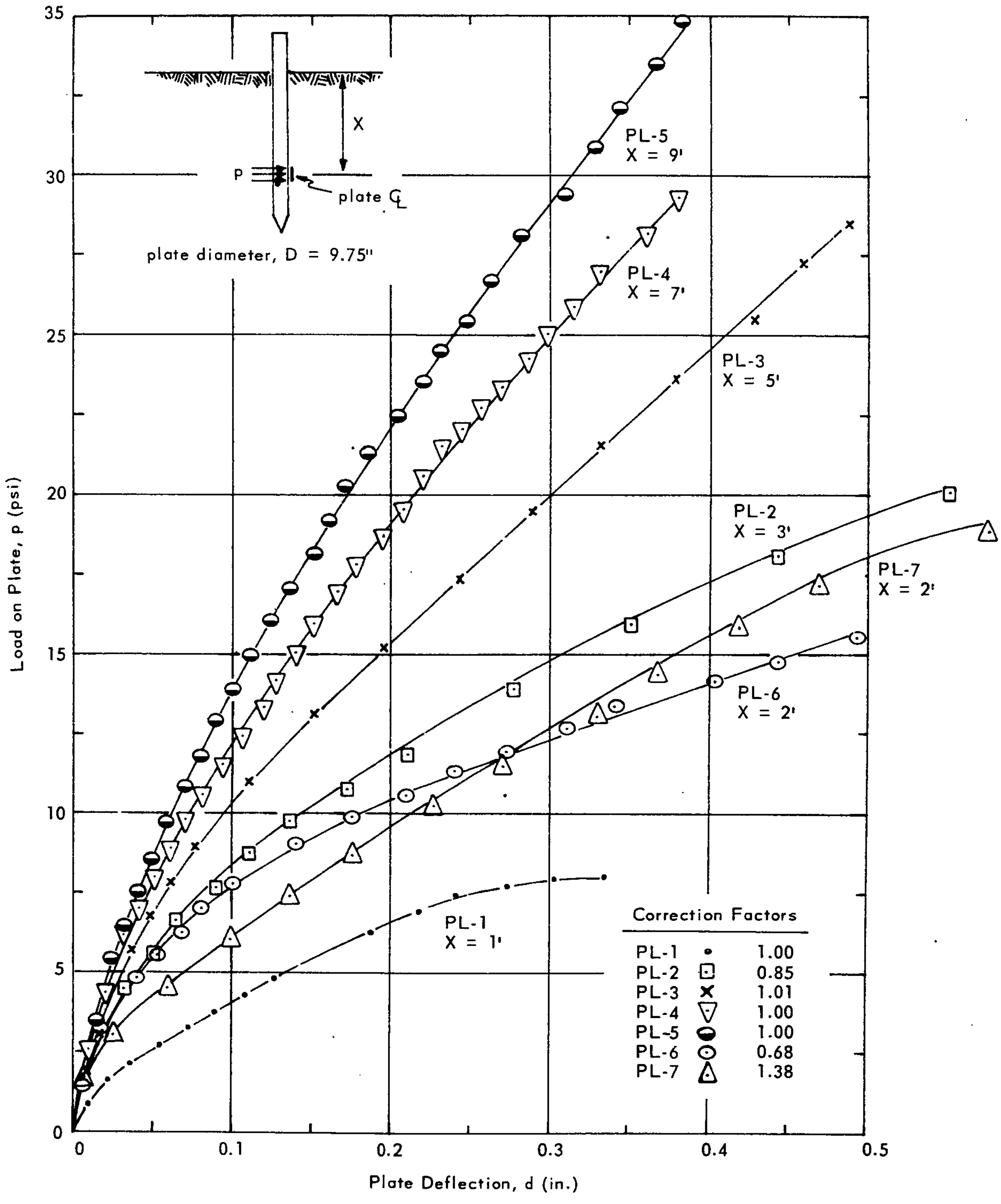


Figure 12. Corrected lateral-plate bearing test data.

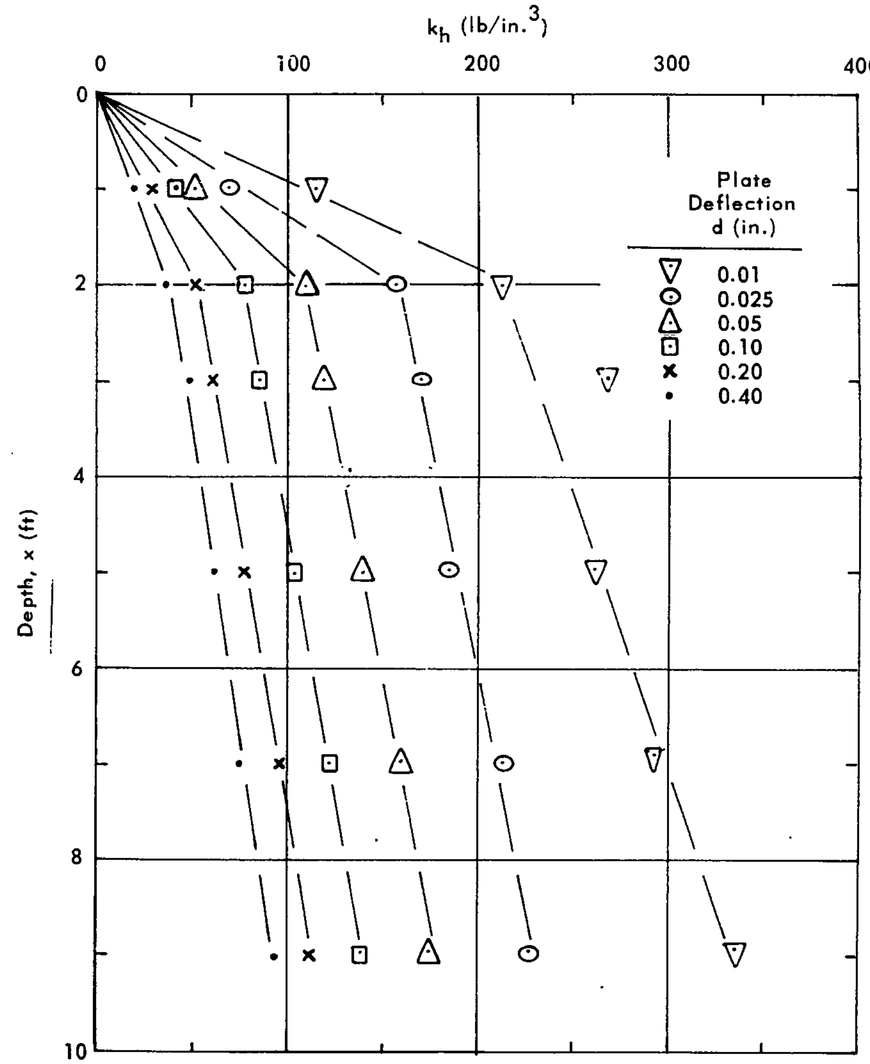


Figure 13. Arithmetic plot of k_h versus depth.

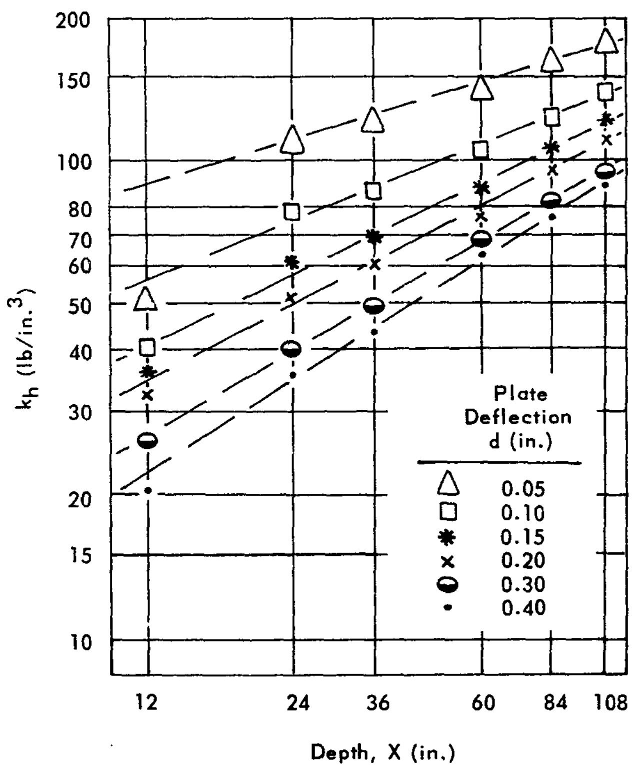


Figure 14. Log-log plot of k_h versus depth.

Equation 5 is shown plotted in Figure 15 and can be compared with the data it is intended to represent in Figure 12. It can be seen that at small deflections, the computed values are somewhat too large, but for deflections greater than 0.10 inch, approximately 1 percent of the plate width, a reasonably good comparison exists. For engineering applications, especially where static loading is concerned, deflections of this magnitude are of most concern to the designer. The behavior of a laterally loaded pile, for instance, depends largely upon the soil within a few pile diameters of the ground surface which is the zone of the greatest pile deflection, making it desirable to have an accurate determination of soil deformation properties at higher lateral displacements.

An interesting feature of Equation 4 is that as plate deflection decreases, depth has a progressively smaller influence on the value of k_h. As d approaches zero, k_h becomes infinitely large, and the depth term X^{S_k} approaches unity. This effect was witnessed from the test data, although it was not as pronounced as that indicated by the equation.

Figure 16 is a plot of the previously described experimental data in a different form to show that the variation of k_h with deflection of the plate is a decreasing exponential function. However, Figure 17 shows that at practical values of deflection (greater than 1 percent of plate width), the rate of change of k_h with deflection is small.

RIGID-PILE TESTS

Description

The piles used in this test program were short lengths of steel pipe with diameters varying from 1 inch to 6 inches. In addition, the 16-inch-wide plate-test apparatus was tested as a rigid pile after completion of the lateral-plate test. The depths of embedment of these piles depended upon the relative stiffness of the pile with respect to the soil. This relationship is expressed as $T = (EI/n_h)^{1/5}$ for granular soils, where E is the modulus of elasticity of the pile material, I denotes the moment of inertia of the pile cross-section, and n_h represents the constant of horizontal subgrade reaction of the soil. If the depth of embedment is less than 2T, a pile undergoes very little flexural strain and can be considered rigid.[9] Therefore, the pile deflection is mainly due to rotation rather than curvature of the pile.

Stiffness determinations were made by simply supporting each pile and applying a point load at midspan. Measurements of the vertical deflection at that point enabled computation of the stiffness, E I, to a suitable degree of accuracy. These stiffness values are reported in the following table.

Pile Width, b (in.)	Stiffness, E I ($lb\text{-}in.^2$)
1	1.32×10^6
2	9.95×10^6
3	5.38×10^7
4	1.41×10^8
6	4.0×10^8
16	1.5×10^9

With the E I values so obtained and with a reasonable estimation of n_h, it was then possible to predict the maximum depth of embedment for each pile to assure that it would be sufficiently rigid.

During the latter portion of the test program, a small metal slab and two layers of Teflon were placed beneath the tip of each pipe, as shown in Figure 4, to reduce horizontal friction at the tip. An attempt was made to accomplish this same goal by pulling the pipes upward slightly, but that process created too much disturbance to the surrounding soil.

Horizontal loads were applied to the piles by means of a 1-inch-diameter cable and a hydraulic ram attached to a D-8 Caterpillar with its blade dropped and brakes set.

Test Program and Procedure

The various rigid-pile tests in a particular setup were performed, starting with the smaller piles and proceeding to the larger. This testing sequence was followed to minimize the soil disturbance associated with testing of the larger piles.

A slightly different loading procedure was used during the pile tests than during the plate test. Whereas the procedure had been to hold the load constant until plate deflection ceased, it was necessary during the pile tests to allow some load reduction while waiting for the deflection to stabilize. It was observed that the load-deflection relationship of the pile was not appreciably affected by this different method of testing.

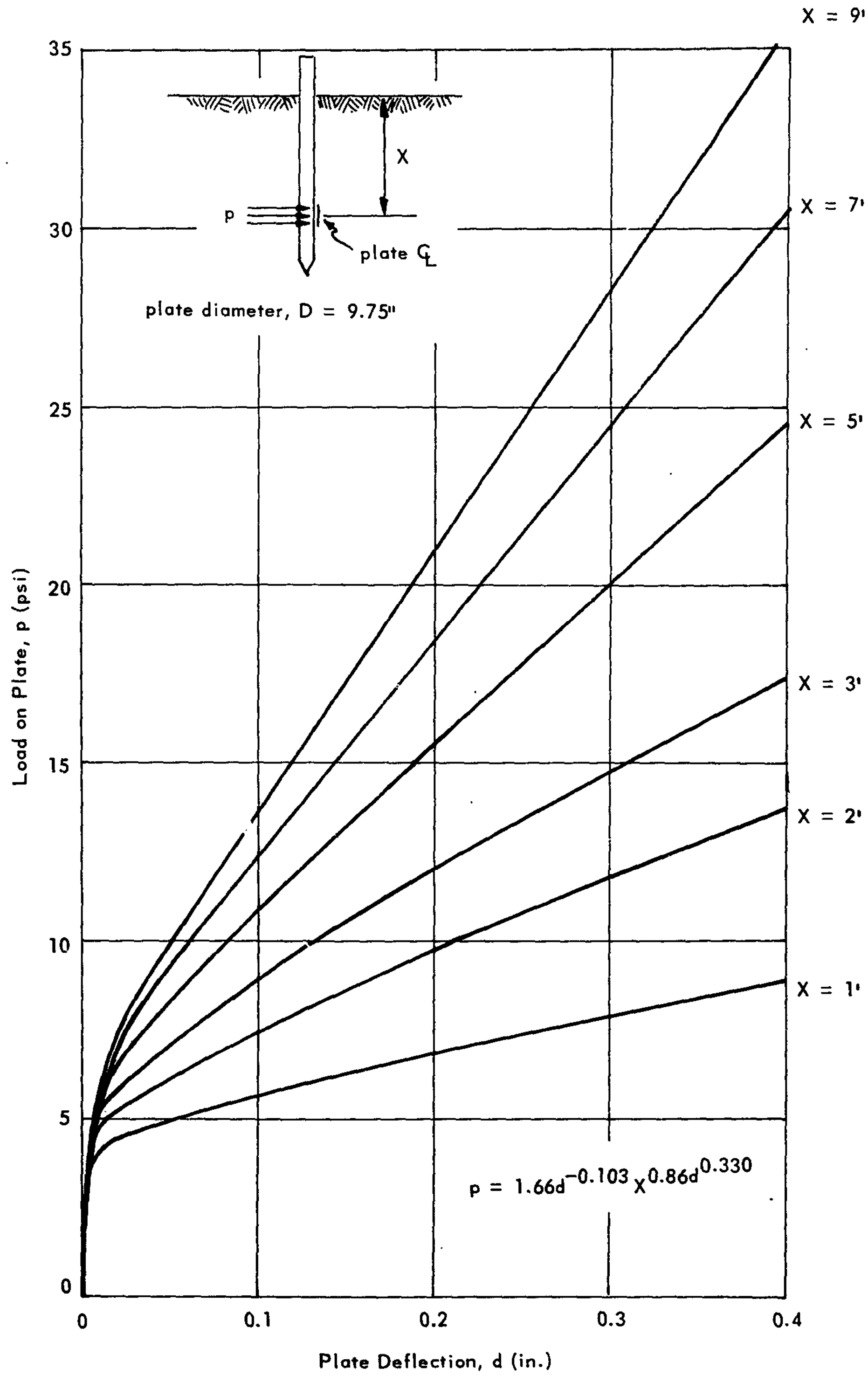


Figure 15. Plot of empirical exponential equation.

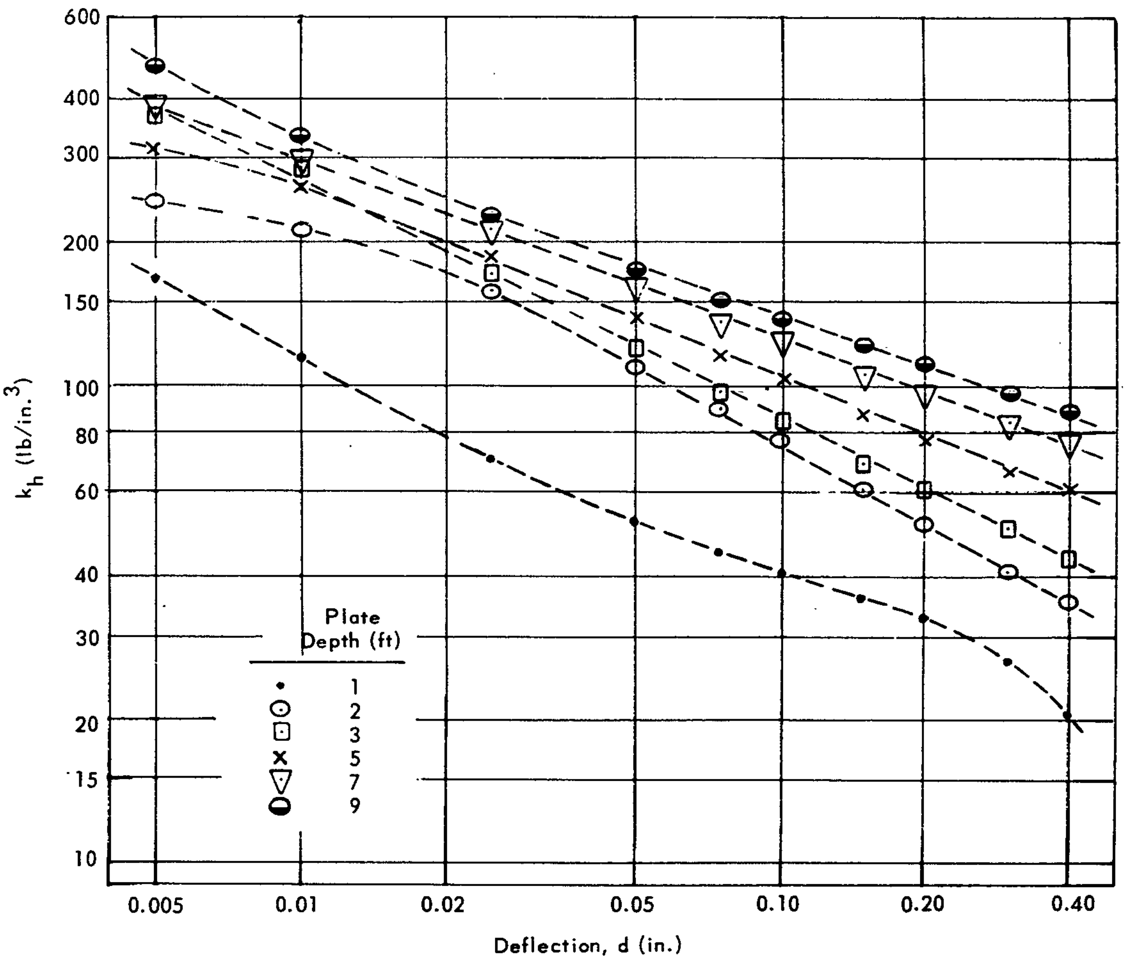


Figure 16. Log-log plot of k_h versus deflection.

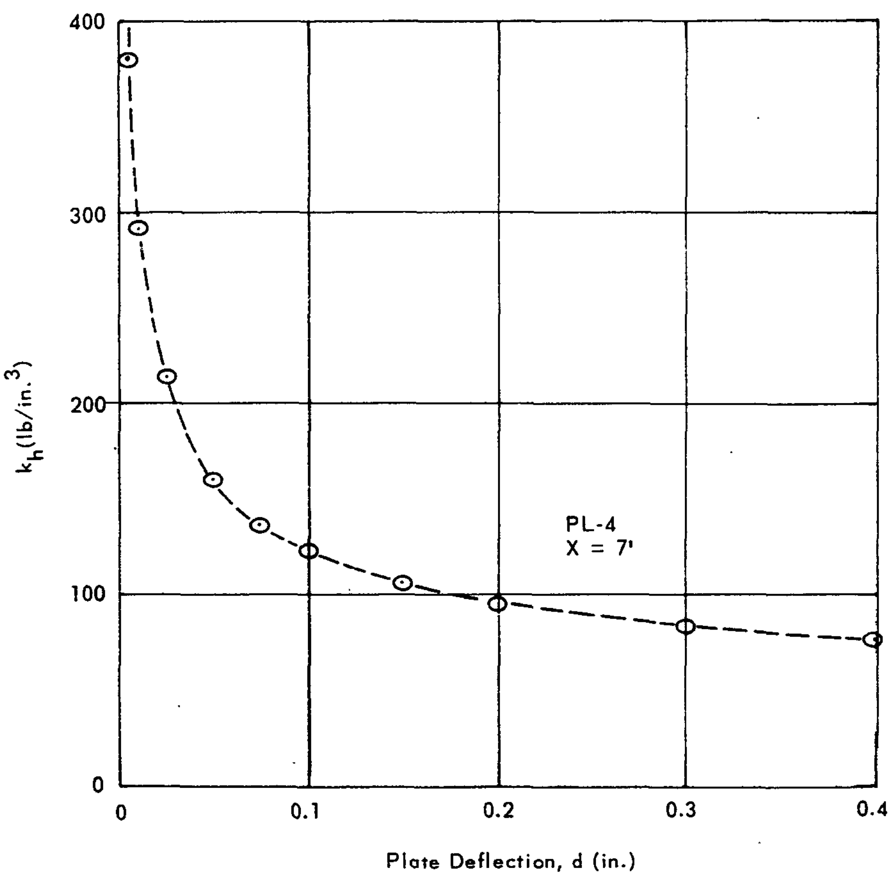


Figure 17. Arithmetic plot of k_h versus deflection.

As was the procedure during the plate test, a load increment was applied and the corresponding deflections were recorded. After 3 to 10 minutes, when the progressive deflection had become negligible, final readings of the horizontal load and pile deflections were taken and the next load increment was applied. The test was stopped when the 2-inch travel capacity of the deflection system was reached.

To obtain a valid rigid-pile test with the lateral-plate apparatus, it was necessary to pull the apparatus in such a direction that the soil previously stressed by the plate would not be required to resist the horizontal motion of the pile. In other words, since the plate was never at a depth greater than D_o (the depth to the point of rotation), the pile was pulled in a direction opposite to the direction of travel of the plate during the earlier plate bearing test. Testing procedures were identical to those during the other rigid-pile tests.

Test Data

Figures 18 through 22 present the results of the 25 rigid-pile tests performed. A description of the piles and the test procedures is contained in an earlier section of this report. The test data are presented in much the same manner as the plate test results, i.e., only the stabilized load-deflection relationships are plotted. With the information shown, the load-slope and the load-y_g (horizontal displacement of the pile at the ground surface) relationships of all the pile tests can be determined. Plots of these relationships are not presented, because a somewhat different approach to the analysis of the data was taken.

As previously mentioned, Reference 8 contains a detailed study of soil-pile behavior. By using equations developed in that paper, one can determine that for a uniform, nonlayered soil deposit, the depth to the point of rotation, D_o, of a rigid pile subjected to a horizontal load at the ground surface should be between 2/3 and 3/4 of the embedded length of the pile, L. It can be seen that D_o/L was greater than 3/4 for tests RP-1 through RP-12, which can be explained by the fact that horizontal friction forces were acting at the pile tips. This horizontal shear was eliminated in the remaining tests and the resulting D_o/L values were less than 3/4.

Design of the pile test program was based upon the need for getting the widest spread of information possible from the six pile sizes while having some tests with different widths of piles at identical embedment depths and test conditions in order to determine the influence of pile width on load-carrying capacity. Because of time limitations, only tests RP-18 and RP-19 were performed during plate tests PL-4 and PL-5, respectively. No pile tests were performed during plate test PL-7 since it was merely a repeat of test PL-6. Photographs taken during pile test RP-14 are presented in Figure 23. The progressive formation of cracks and a bulging, bulb-shaped wedge in front of the pile were typical for every test.

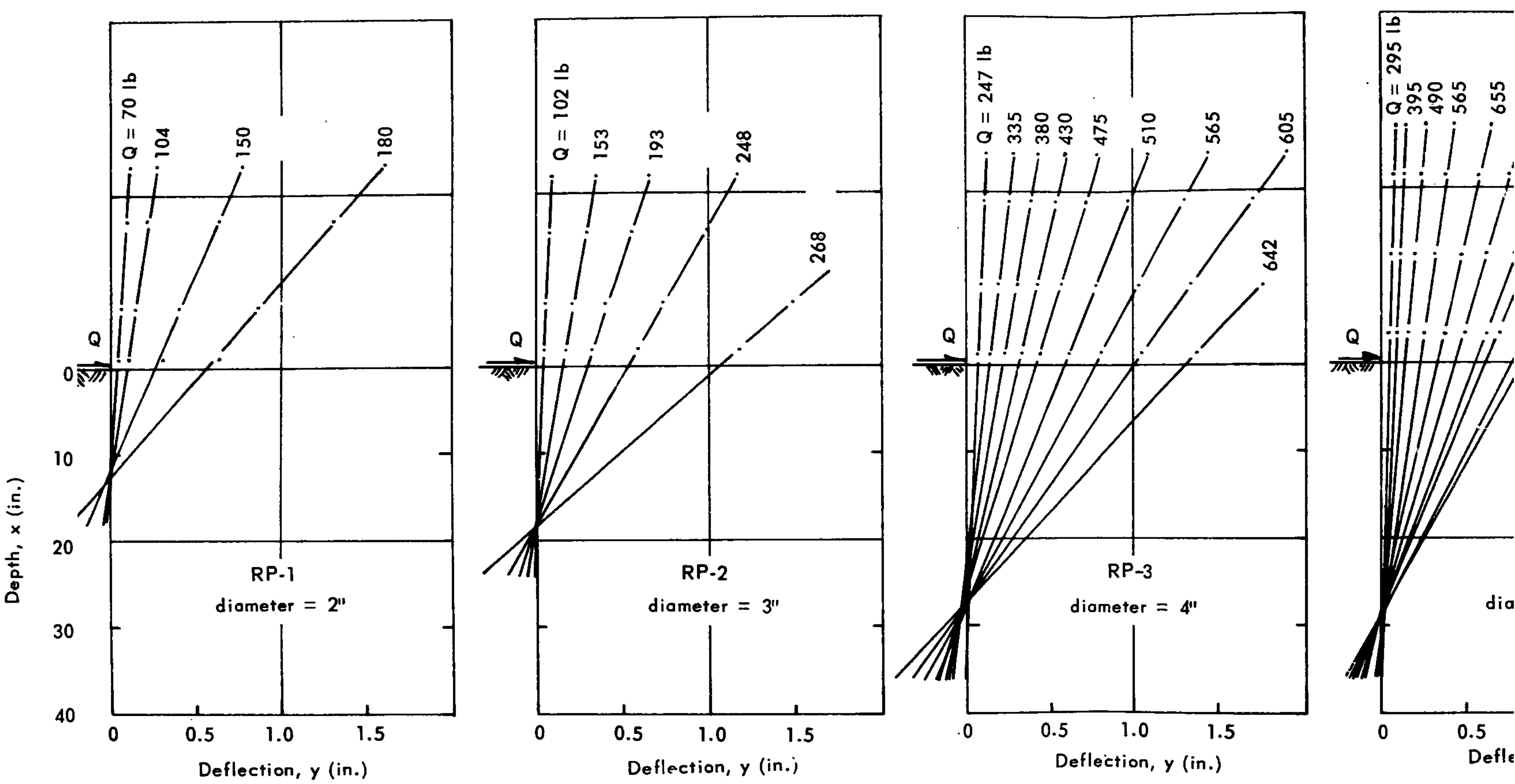
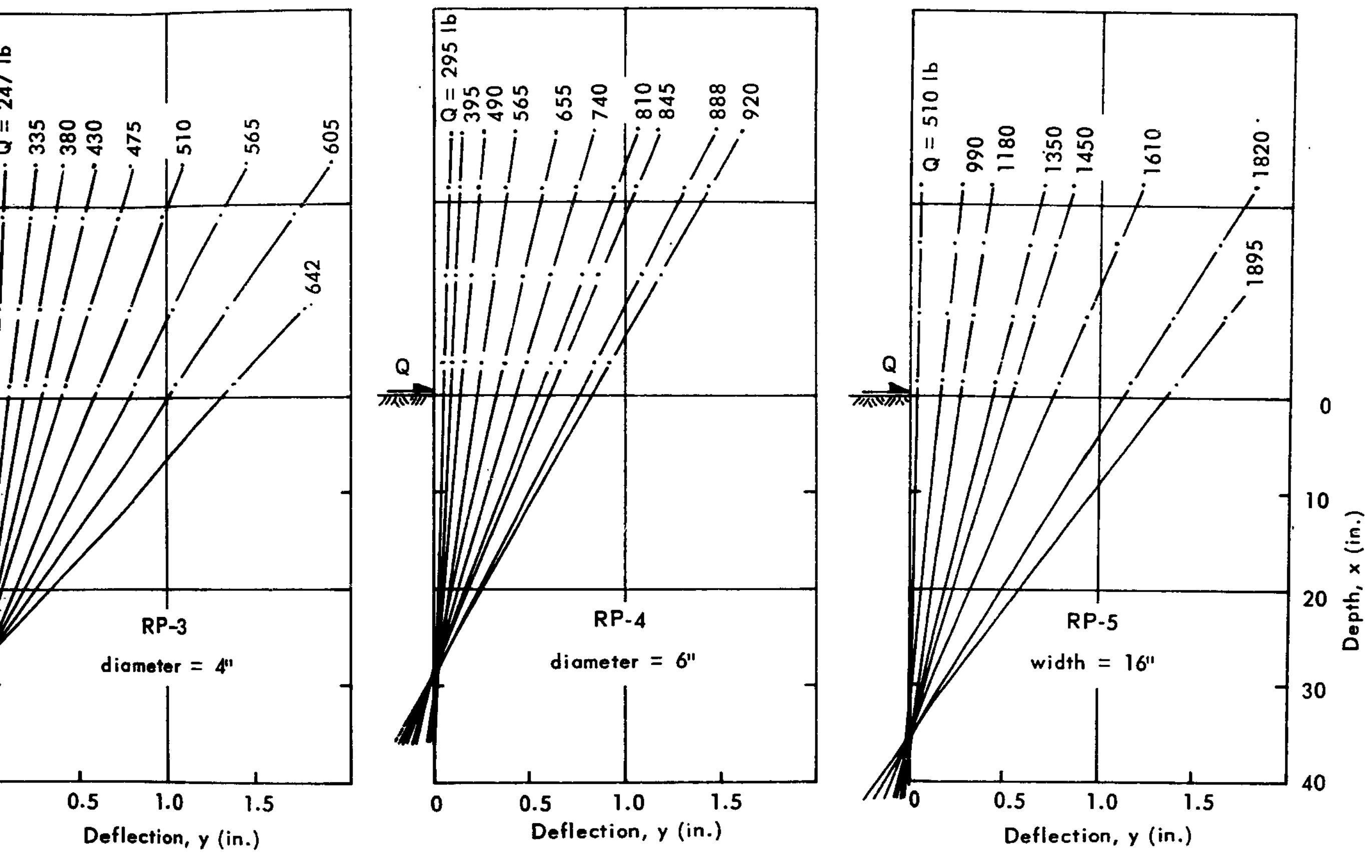


Figure 18. Pile-test data, RP-1 through RP-5.

2



Pile-test data, RP-1 through RP-5.

1

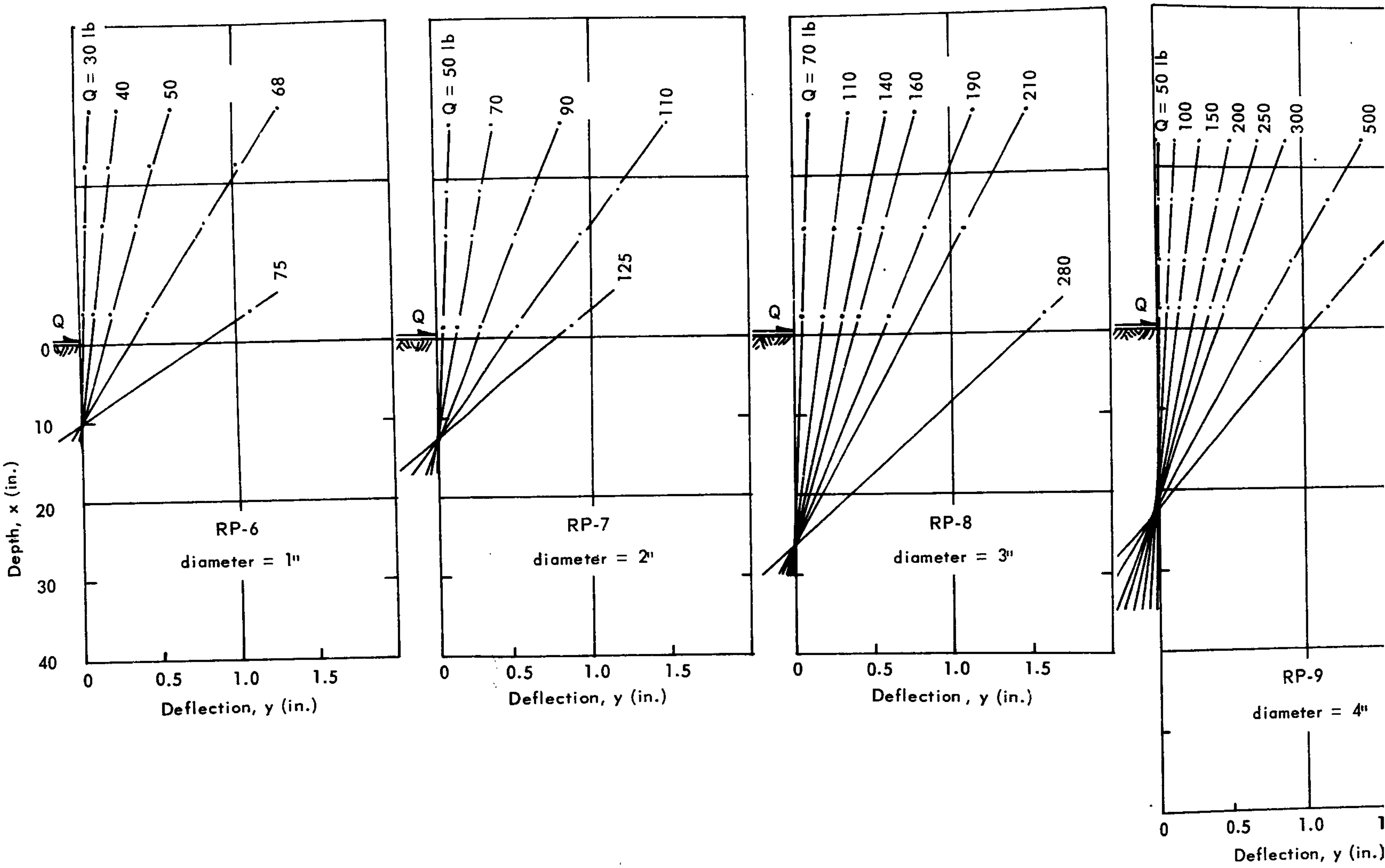
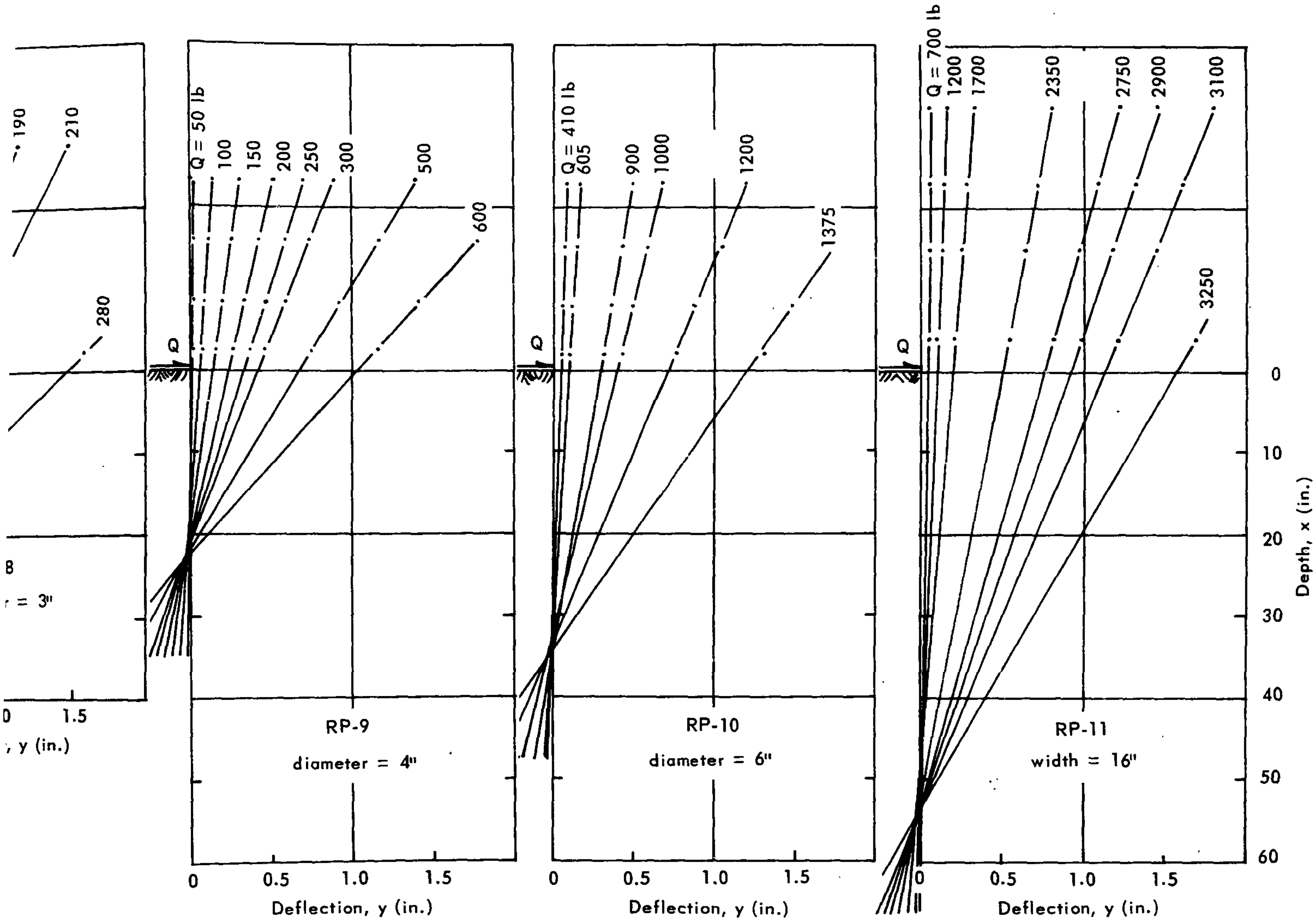


Figure 19. Pile-test data, RP-6 through RP-11.

2



!. Pile-test data, RP-6 through RP-11.

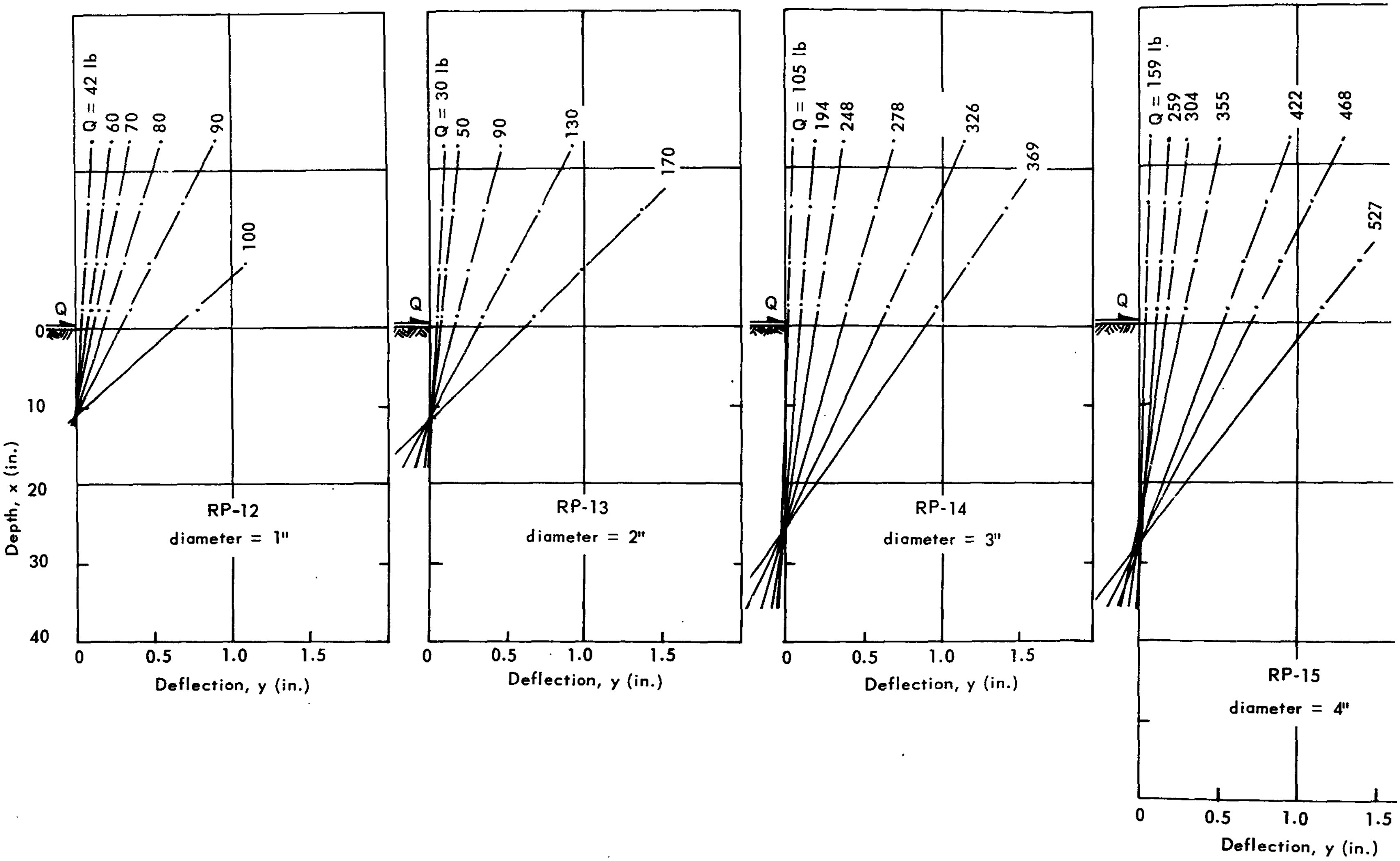
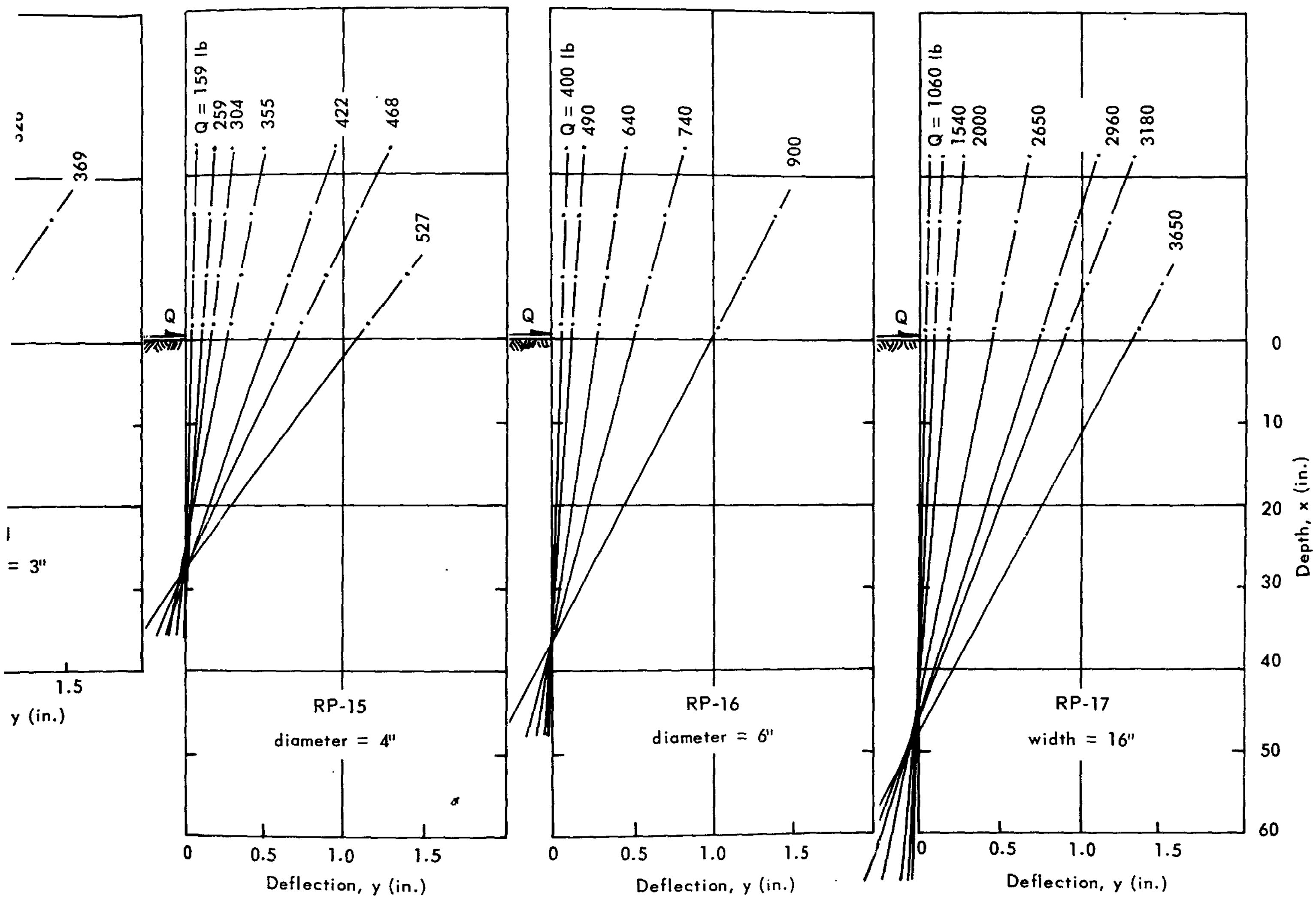


Figure 20. Pile-test data, RP-12 through RP-17.

2



). Pile-test data, RP-12 through RP-17.

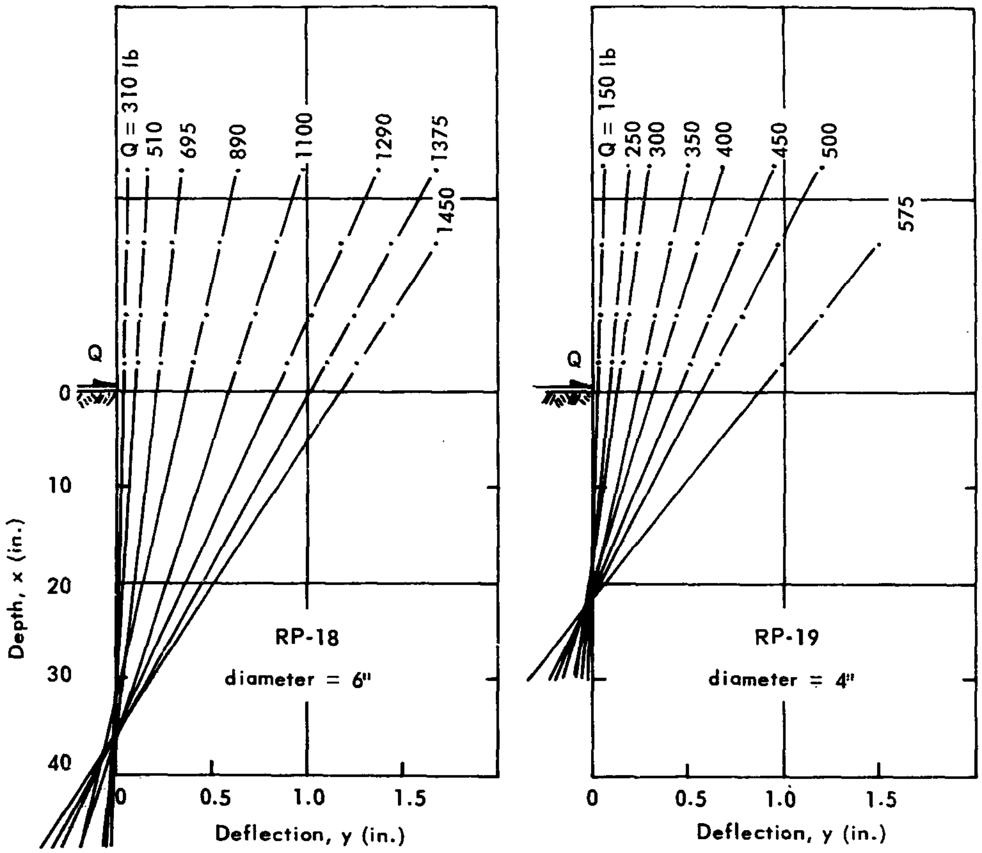


Figure 21. Pile-test data, RP-18 and RP-19.

1

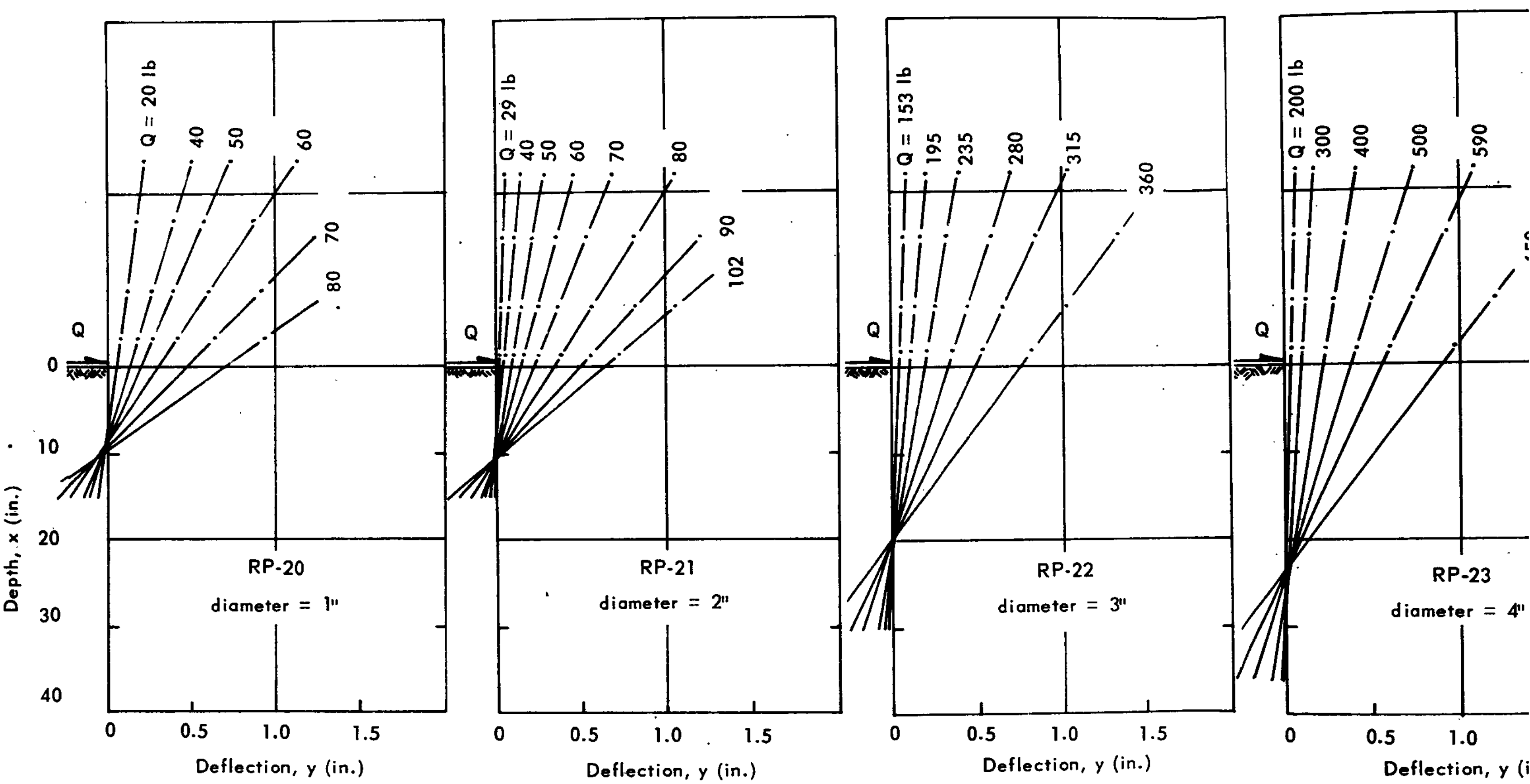
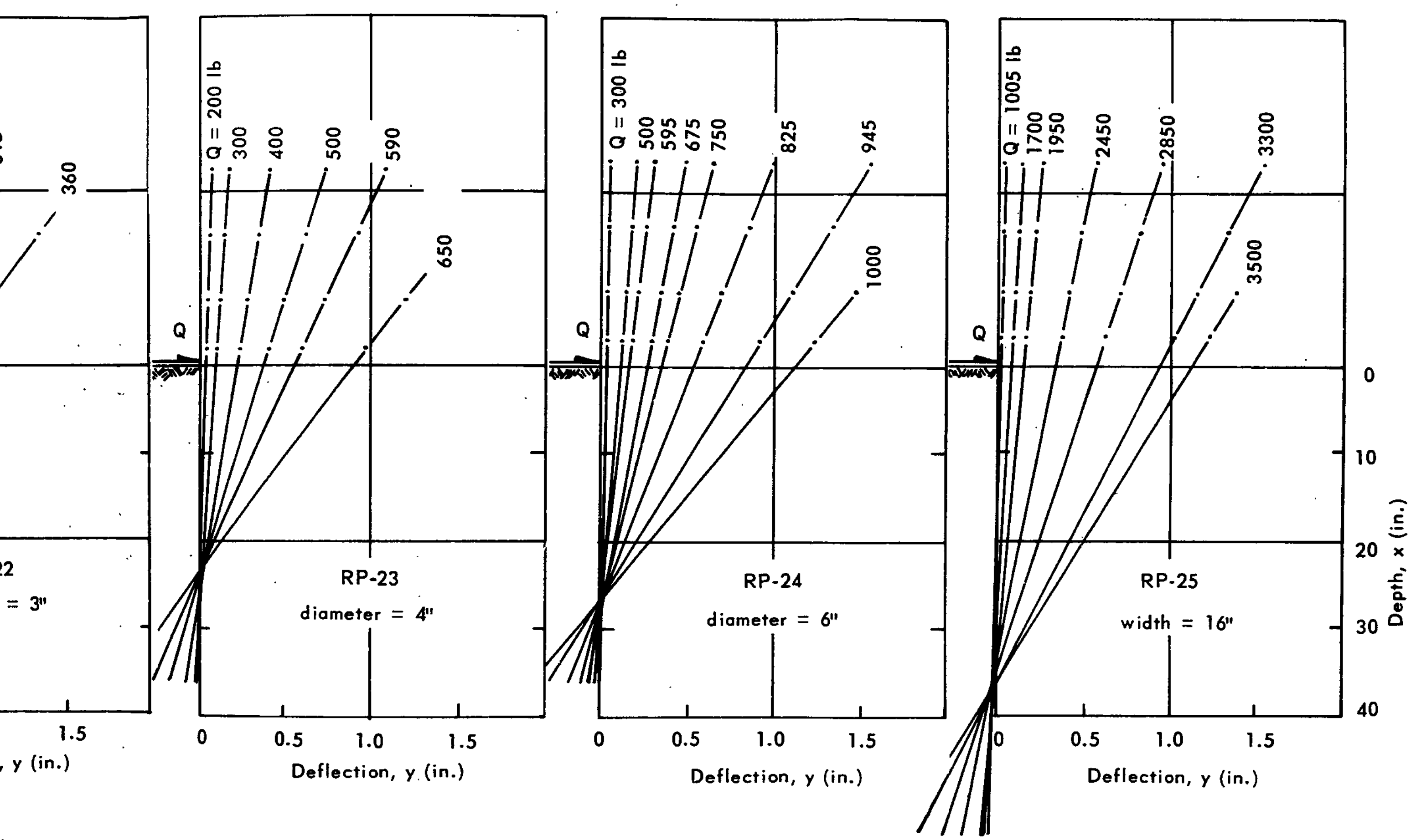


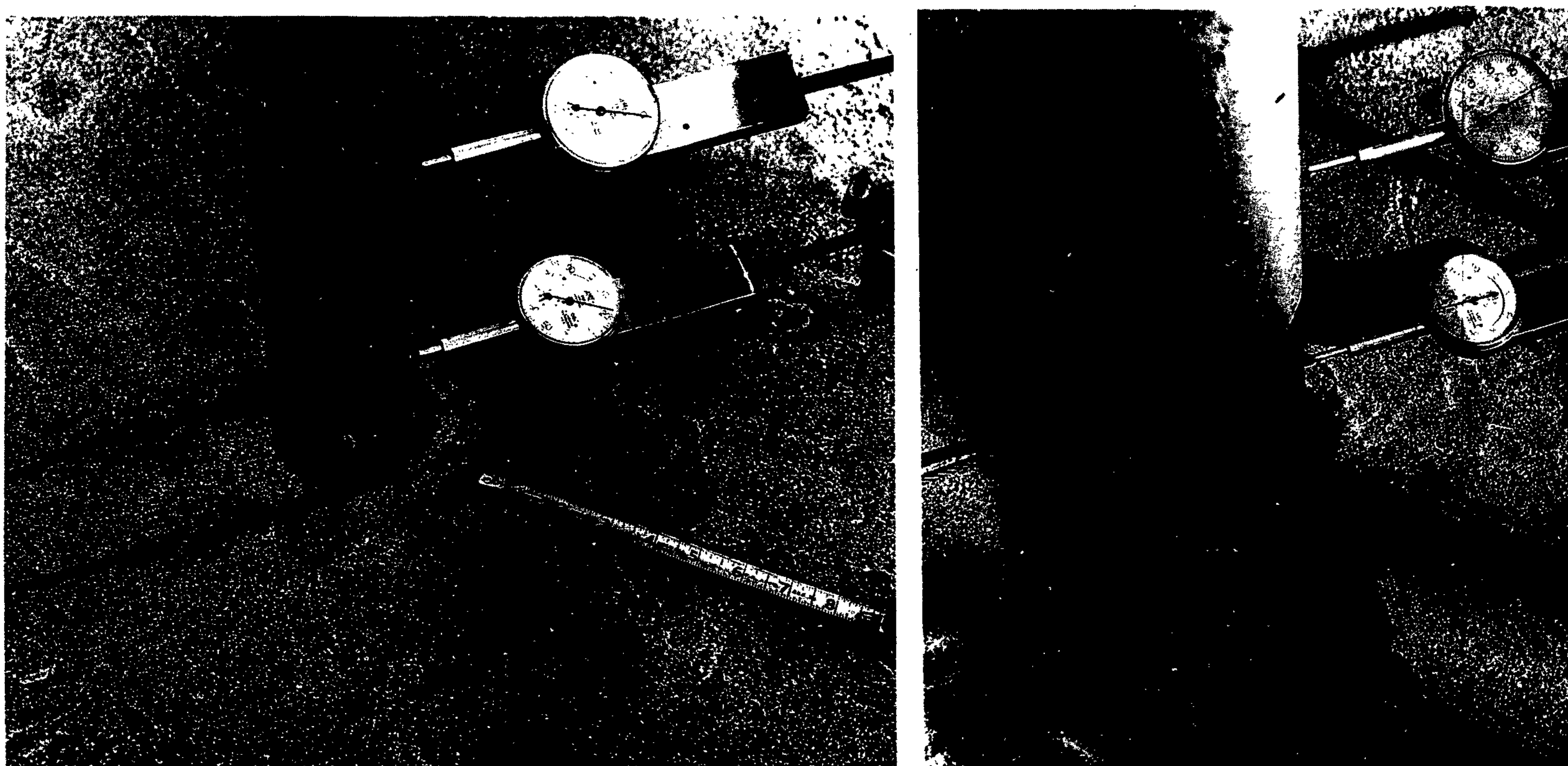
Figure 22. Pile-test data, RP-20 through RP-2

2



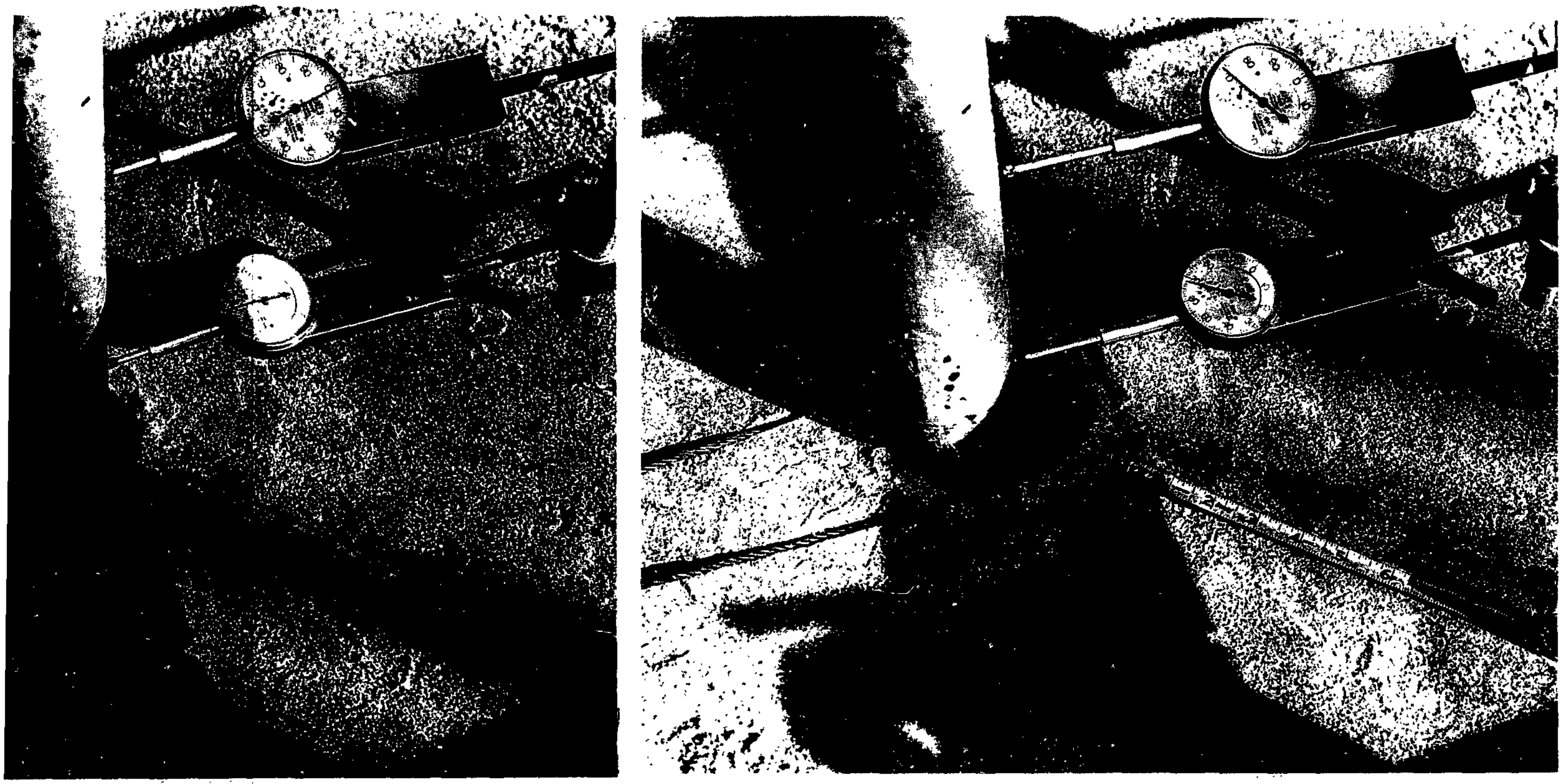
Pile-test data, RP-20 through RP-25.

1



(a) Q = 278 lb

Figure 23. Crack patterns during RP-1



(b) Q = 326 lb

(c) Q = 408 lb

3. Crack patterns during RP-14.

Data Analysis

Analysis of the pile-test data was based upon the concept of a constant of horizontal subgrade reaction,[1] n_h. This term assumes a linearly increasing soil modulus with depth and is comparable, if shape effects are ignored, to the k_h term obtained during the plate tests, multiplied by the plate diameter and divided by depth. The resulting units are force divided by length cubed (lb/in.^{3}).

Although the plate tests showed that soil modulus varied approximately exponentially with depth in moist beach sand, Figure 13 shows that at shallow depths, the zone where most of the soil resistance to pile movement is developed, k_h increases essentially linearly with depth. This distribution of soil strength with depth is also substantiated by the vane shearing test data and, as discussed earlier, can be explained by the presence of apparent cohesion in the soil. Because of the shallow depths of embedment of the piles, it was assumed for purposes of analysis that the variation of soil modulus with depth could be represented by some value of n_h.

With this simplified assumption, it was still necessary to determine the variation of n_h with pile deflection and width. The approach taken was to use the equation for y_g as a function of horizontal load at the groundline, Q; depth of embedment, L; and constant of horizontal subgrade reaction, n_h.[8]

$$y_g = \frac{18Q}{n_h L^2} \tag{6}$$

It can be seen that pile width, b, does not enter this equation, because an assumption involved in the term n_h is that it is a constant for any width of pile.[1] (It is understood that this assumption is not valid for very narrow piles where a punching-type action or lateral bearing failure occurs.[8]) However, it will be shown later that n_h is a function of pile width for the type of soil considered in this analysis.

One can enter Equation 6 with a given rigid-pile length and applied horizontal load to compute the value of n_h necessary to allow any value of deflection, y_g. This was done for the data in Figures 18 through 22 and a log-log plot of n_h versus y_g was made for each plate test. As was found from the plate-test data, the variation of soil resistance with deflection could be approximated by an exponential function. However, it was seen that the data for the wider piles generally fell above the lines corresponding to the narrower piles, meaning that n_h is some function of pile width, b, for this type of soil.

The assumption of n_h constant with pile width requires that the zone of soil stressed in front of the pile be directly proportional to the pile width and also requires that the slope, E_s, of the soil stress-strain curve be constant in the range of stress considered. It was shown by the plate tests that k_h decreases with increasing deflection, so it is probable that E_s also decreases with increasing deflection, especially at shallow depths and higher deflections. This reduction of k_h could be a result of a longer zone of soil in front of the plate being stressed, which would have the same effect as a reduction in E_s. Probably both have an influence on the reduction in k_h with deflection, but for this analysis it will be assumed that a reduction in E_s exists which is proportional to the reduction in k_h with deflection. This means that for uniform soil conditions, n_h is a constant function of y_g/b — pile deflection divided by pile width — rather than just a function of y_g. Likewise, the factor k_h multiplied by plate width (corresponding to n_h) should also be a constant function of deflection divided by plate width. A clearer example of this can be shown by the cross-sectional view of two loaded areas of different width shown in Figure 24. With all units in pounds and inches, and considering a 1-inch strip of loaded area, the following relationships can be obtained:

If $d_2 = n d_1$ and $b_2 = n b_1$, then

$$\frac{d_1}{b_1} = \frac{n d_1}{n b_1} = \frac{d_2}{b_2}$$

and $k_{h_1}(b_1)$ must equal $k_{h_2}(b_2)$.

$$k_{h_1}(b_1) = \frac{Q_1}{(b_1)(1'')(d_1)}(b_1) = k_{h_2}(b_2) = \frac{Q_2}{(b_2)(1'')(d_2)}(b_2) \qquad (\text{lb/in.}^2)$$

if $Q_1/d_1 = Q_2/n d_1$.

Therefore,

$$Q_2 = n Q_1 \qquad (\text{lb})$$

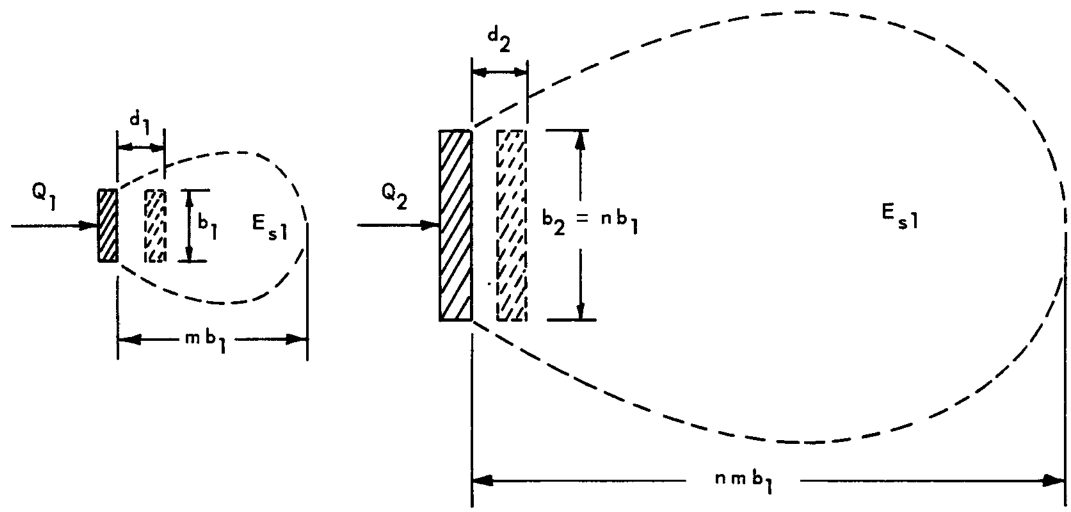


Figure 24. Effect of width of loaded area.

and the state of strain in the soil is the same for both widths.

$$E_{s1} \cong \frac{\dfrac{Q_1}{b_1\,(1'')}}{\dfrac{d_1}{mb_1}} = E_{s2} \cong \frac{\dfrac{nQ_1}{nb_1\,(1'')}}{\dfrac{nd_1}{nmb_1}} \qquad \text{(psi)}$$

However, if d_2 is equal to d_1 and $b_2 = nb_1$,

$$k_{h_1}(b_1) = \frac{Q_1}{(b_1)(1'')(d_1)}(b_1) = k_{h_2}(b_2) = \frac{Q_2}{(nb_1)(1'')(d_1)}(nb_1) \qquad (\text{lb/in.}^2)$$

which requires, $Q_1 = Q_2$.

But if $d_2 = d_1$, d_1/b_1 is not equal to d_2/b_2 and $k_{h_1}(b_1)$ cannot equal $k_{h_2}(b_2)$;

therefore, Q_1 cannot equal Q_2.

In addition,

$$E_{s1} = \frac{\dfrac{Q_1}{b_1\,(1'')}}{\dfrac{d_1}{m b_1}} = E_{s2} = \frac{\dfrac{Q_1}{n b_1\,(1'')}}{\dfrac{d_1}{n m b_1}} \qquad \text{(psi)}$$

a condition which cannot exist unless $d_1/b_1 = d_2/b_2$.

This shows that both n_h and $k_h\,(b)$ should be expressed as functions of deflection divided by width at a given depth in this type of soil.

A typical log-log plot of the n_h values computed from the pile tests versus the corresponding deflection factor, y_g/b, is shown in Figure 25. Again, the decreasing soil strength with increasing deflection is shown. For each test setup there was a fairly close grouping of the data for pile widths from 3 to 16 inches. Throughout the test program, the results of tests using the 1- and 2-inch piles indicated that a large portion of their load-carrying capacity resulted from shearing forces at the sides of the piles, making the computed values of n_h too large. As a result, these smaller piles were not considered in the following analysis.

The data presented in Figure 25 have been plotted arithmetically in Figure 26 to show that at large deflections the rate of change of n_h with deflection is small. It is of practical interest to note that the error involved in the assumption of n_h constant with deflection at large deflections is small, particularly since the moment in a pile varies inversely with the fifth root of n_h and deflection varies inversely with the three-fifths power of n_h. Small errors in n_h will have very little influence on the accuracy of moment and deflection computations. However, at small deflections the variation of n_h with deflection is significant and width of loaded area must be taken into account by considering deflection in terms of pile width.

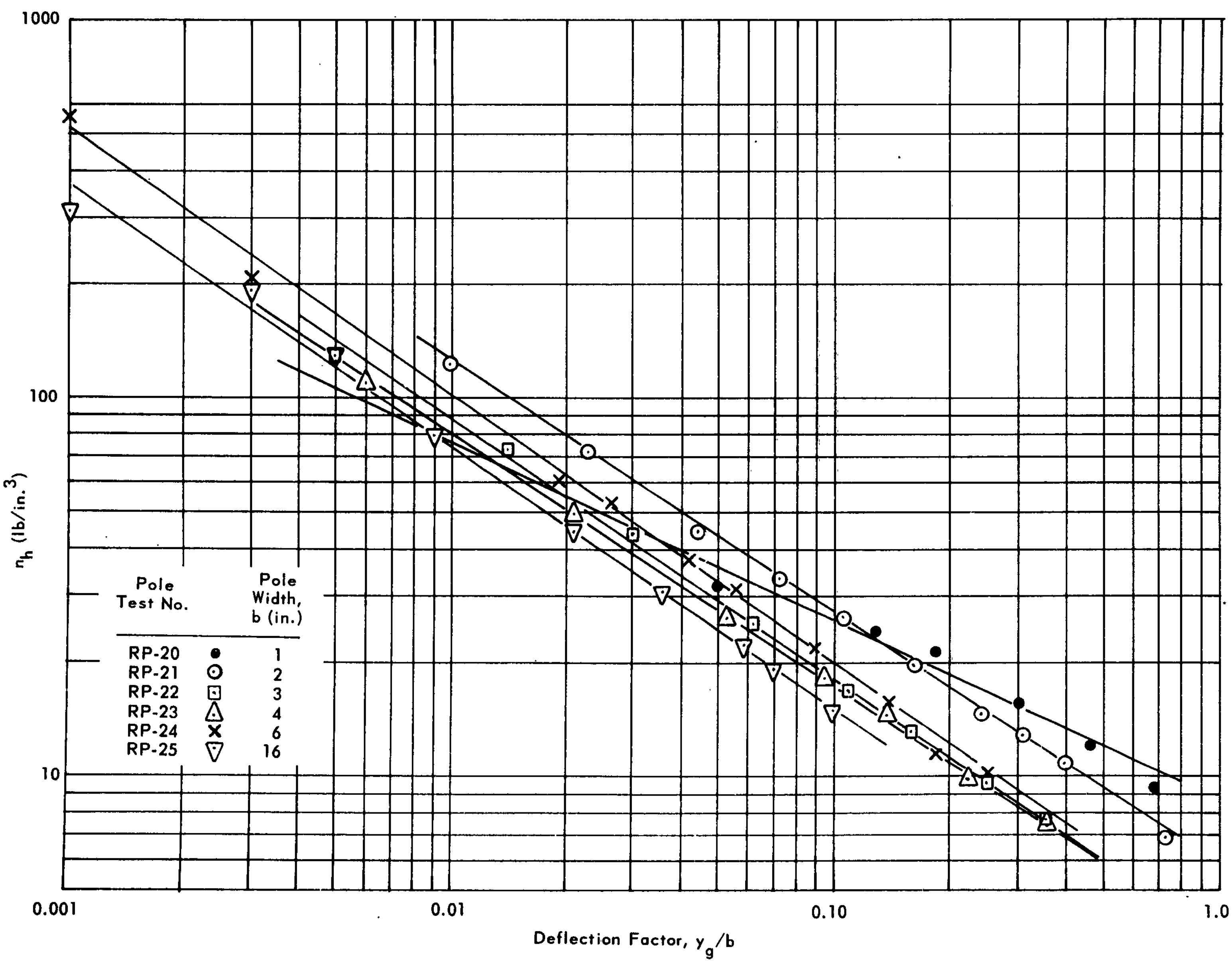


Figure 25. Variation of n_h with pole deflection for PL-6.

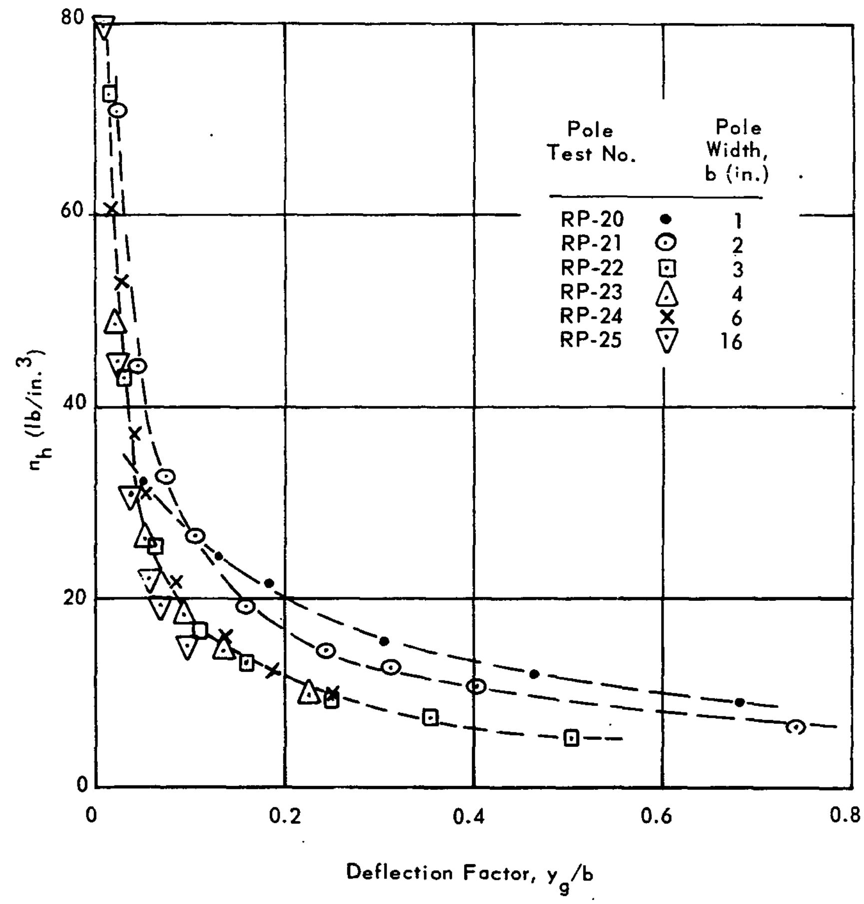


Figure 26. Arithmetic plot of n_h versus pile deflection.

A comparison of the data in Figure 26 with similar data from the plate tests in Figure 17 reveals that the curve shapes are similar, but there is a significant difference in the deflection at which the major slope change occurs. For the plate test, this transition came at a deflection of approximately 0.1 inch or 1 percent of the plate width. For the pile tests, the corresponding transition occurred at a groundline deflection of about 10 percent of pile width. A full understanding of this difference must await further research, but there are three possible explanations which bear investigation: (1) shearing forces at the sides of the piles may account for some of the difference; (2) the difference in the response of circular- and strip-shaped loaded areas may be a contributing factor; and (3) it is very likely that most of the difference is due to the nature of the response of a pile loaded horizontally at the groundline in that it undergoes more displacement near the ground surface than at any other point on the pile. Since n_h is a measure of the variation of soil modulus with depth, and soil modulus decreases with deflection, the modulus of the weaker soil near the surface changes rapidly with deflection while the modulus of the deeper and stiffer soil changes more slowly. The result is that n_h varies less with deflection than does k_h.

Exponential equations of the form

$$n_h = c_1 \left(\frac{y_g}{b}\right)^{c_2} \qquad (7)$$

can be written for the straight lines shown in Figure 25. For any one test setup in the experimental program, all tests resulted in essentially the same value of c_2 (excluding the two narrowest piles), but there was some variation in the values of c_1. There were no indications of any influence of pile size on c_1. The values of c_2 and ranges of c_1 obtained during the pile-test program are listed in Table II.

A study of the vane shearing strength data in Figures 8 through 10 failed to establish any exact relationship between S and the values of c_1 and c_2, but it does appear that increasing soil strength increases the value of c_2 to some extent, while having little effect on c_1. This means that increasing the shearing strength of the soil has the effect of increasing n_h at small deflections, but does not influence n_h appreciably at large deflections. It appears that the equation which most nearly corresponds to the typical soil conditions during the test program can be written as

$$n_h = 3.5 \left(\frac{y_g}{b}\right)^{-0.68}$$

Table II. Range of Factors for Equation 7

(Data from 1- and 2-inch-diameter piles not included)

Pile Test Numbers	Range of c_1	Value of c_2
RP-2 through RP-5	2.9-4.4	-0.66
RP-8 through RP-11 1/	—	—
RP-14 through RP-17	2.1-2.8	-0.68
RP-18	3.8	-0.60
RP-19	4.8	-0.63
RP-22 through RP-25	3.1-4.0	-0.69

1/ Results were erratic due to friction at pile tip.

Results of these plate tests and pile tests have shown that k_h decreases exponentially with increasing deflection in moist beach sand. The pile test results have further shown the influence of width of loaded area on k_h, indicating that Equation 4 should be redetermined as

$$k_h(D) = C_1 (d/D)^{-1.103} X^{C_2 (d/D)^{0.330}}$$

where D is the plate diameter and the terms C_1 and C_2 are emperically determined constants. These constants can easily be determined by the procedures used in determining Equation 4, but further research will be needed to firmly establish this relationship and to determine if similar relationships exist for other types of soil.

FINDINGS

The following findings are based upon the results of the tests reported herein considering only one soil type and condition. Further experimentation will be needed to establish the validity of these findings for other soil types.

1. In a soil deposit with in situ shearing strength constant or increasing with depth, lateral bearing capacity and horizontal modulus of subgrade reaction, k_h, increase with depth.

2. Lateral bearing capacity and k_h at any depth are approximately directly proportional to the shearing strength of the soil at that depth.

3. It was shown by the plate tests that k_h is an exponential function of depth, X, and deflection, d. For the tests in this series, the equation is

$$k_h = 1.66d^{-1.103} X^{0.86d^{0.330}}$$

and is valid for deflections greater than 1 percent of the plate width.

4. As plate deflection decreases, depth has a progressively smaller influence on k_h.

5. At shallow depths, k_h can be assumed to increase linearly with depth for purposes of analysis. Therefore, values of a constant of horizontal subgrade reaction, n_h, can be computed from the pile tests.

6. The values of n_h computed from the pile tests decrease exponentially with deflection of the pile.

7. Because of shear forces at the sides of the piles, the soil-strength characteristics computed from tests of very small piles (1- and 2-inch diameters) were much higher than those calculated from tests of the larger piles. Therefore, the results of those tests were ignored in the following findings.

8. The magnitude of n_h was found to be essentially a constant function of the factor y_g/b, deflection divided by width. In other words, a pile of width b_1 deflected a distance y_{g_1} would indicate the same value of n_h as a pile of different width n_{b_1} and deflection $n\,y_{g_1}$, if they both were tested in the same soil. However, if both piles were deflected a distance y_{g_1}, the wider pile would indicate a much higher value of n_h. For the average soil conditions during this program, the variation of n_h with deflection and width can be represented by $n_h = 3.5\,(y_g/b)^{-0.68}$.

9. The preceding finding indicates that the equation for k_h (given in finding number 3) could be rewritten in terms of k_h times plate diameter as a function of depth, X, and deflection divided by diameter, d/D, $[k_h (D) = f(X, d/D)]$ to account for varying plate size.

FUTURE STUDIES

An extension of this test program is anticipated in which a new type of lateral-plate test device, currently under design, will be used. This new device will consist of a 12-inch-diameter steel pipe which can be driven into a soil deposit in three separate segments. After the soil is removed from the interior of the segmented pipe, a loading mechanism can be inserted to measure the lateral load-deflection relationship of the middle segment (from 4 to 12 inches long) while reacting against the upper and lower segments. Advantages of this type of device include its ability to measure the horizontal modulus of natural soil deposits without creating undue disturbance during test preparation, and its close simulation of the soil stress conditions associated with a laterally loaded pile. The data obtained with this device will be useful not only for the design of laterally loaded piles, but also for other practical applications such as the design of retaining walls, anchored bulkheads, buried structures, buried conduits, etc. Further soil moduli studies will be made including the effects of size of loaded area, repetitive loading, magnitude of deflection, and depth in various types of soil deposits using this device in conjunction with rigid-pile tests and soil property determinations.

ACKNOWLEDGMENTS

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Dr. M. T. Davisson of the University of Illinois reviewed this report under a consulting services contract with NCEL.

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LIST OF SYMBOLS

a	Relative depth of surface layer of soil
b	Pile width
C, c	Dimensionless constants
D	Plate diameter
D_o	Depth to the point of rotation of a pile
D_r	Relative density of soil
d	Plate deflection
E	Modulus of elasticity
E_s	Modulus of elasticity of soil
G	Specific gravity of soil
I	Moment of inertia of pile cross-section
I_k, i_i, i_s	Intercept factors
k	Modulus of horizontal subgrade reaction
k_h	Coefficient of horizontal subgrade reaction
L	Embedded length of a pile
m	Dimensionless constant
m_h	Ratio between coefficient of horizontal subgrade reaction, k_h, and depth below surface
N	Standard penetration
n	Dimensionless constant
n_h	Constant of horizontal subgrade reaction

p	Average soil pressure on plate
Q	Horizontal load on pile at groundline
S	In situ vane shearing strength
S_k, s_i, s_s	Slope factors
T	Relative stiffness factor
t	Time
w	Moisture content
X	Depth to centerline of plate
X_1	Depth of surface layer of soil
x	Depth below ground surface
y	Horizontal pile deflection
y_g	Horizontal pile deflection at groundline
γ_d	Dry density of soil

Appendix

RIGID PILES IN A LAYERED SOIL SYSTEM

Although subgrade modulus has been shown in the previous sections to vary in a complex manner with depth and deflection, it may be possible, for small projects, to estimate a reasonable variation of modulus with depth for some limiting value of deflection. The modulus term to be referred to in this discussion is called the modulus of horizontal subgrade reaction, k, in units of lb/in.^{2} and is the result of multiplying the coefficient of horizontal subgrade reaction, k_h, by the pile width. It may be possible to approximate the variation of k with depth by a layered system as shown in Figure 27. With these approximations and estimations, the equation for the deflection of a rigid pile can be shown to be

$$\frac{k_0 L y_g}{Q} = \frac{1}{[(C-1)a+1] - \dfrac{3[(C-1)a^2+1]}{4[(C-1)a^3+1]}} \tag{8}$$

where the terms are defined as in Figure 27. Likewise, the point of rotation of a rigid pile can be computed from the following relationship:

$$\frac{D_o}{L} = \frac{2[(C-1)a^3+1]}{3[(C-1)a^2+1]} \tag{9}$$

Equation 8 is plotted in Figure 27, and it can be seen that a relatively shallow surface layer exerts considerable influence on the behavior of a rigid pile.

In some instances, a soil profile may be found which cannot be approximated by a two-layer system, and a variation of k with depth as shown in Figure 28 may be a closer representation. This becomes evident when one investigates the typical variation of shearing strength with depth shown in Figure 11 and the variation of k_h with depth plotted in Figure 13. The deflection and rotation equations for this condition become

$$\frac{k_0 L y_g}{Q} = \frac{1}{1 - \frac{a}{2} + \frac{(a^2 - 3)^2}{3(a^3 - 4)}} \tag{10}$$

and

$$\frac{D_o}{L} = \frac{a^3 - 4}{2(a^2 - 3)} \tag{11}$$

respectively. These equations were not used in the rigid-pile analysis because a study indicated that due to the large values of X_1/L during the tests, the deflections computed by assuming k increasing linearly with depth were not appreciably different from those computed by assuming k constant below a depth X_1. However, a comparison of a plot of Equation 10 in Figure 28 with Figure 27 shows the errors which can result from a misrepresentation of the subgrade modulus at shallow depths.

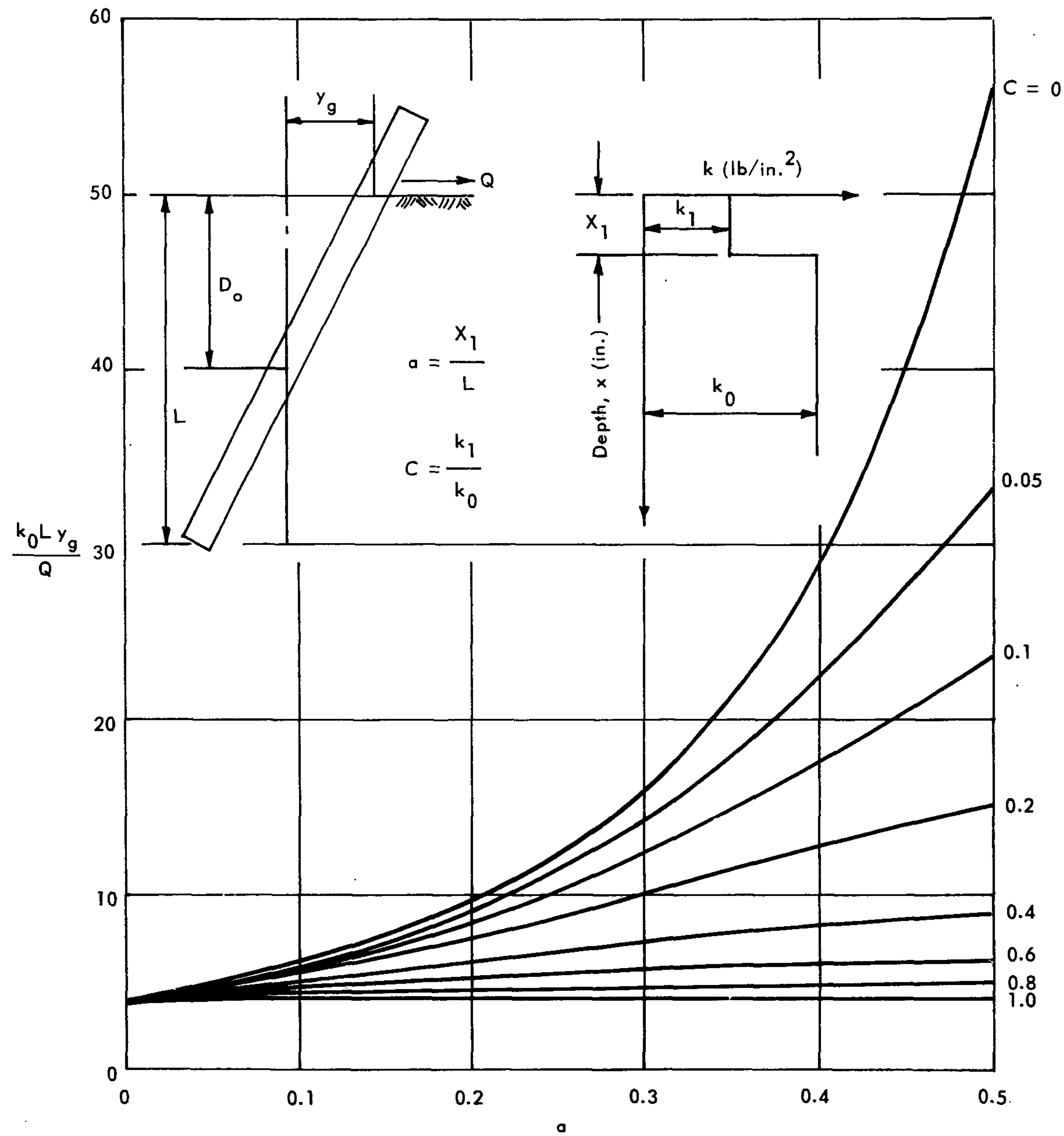

Figure 27. Pile-deflection parameter for layered soil system.

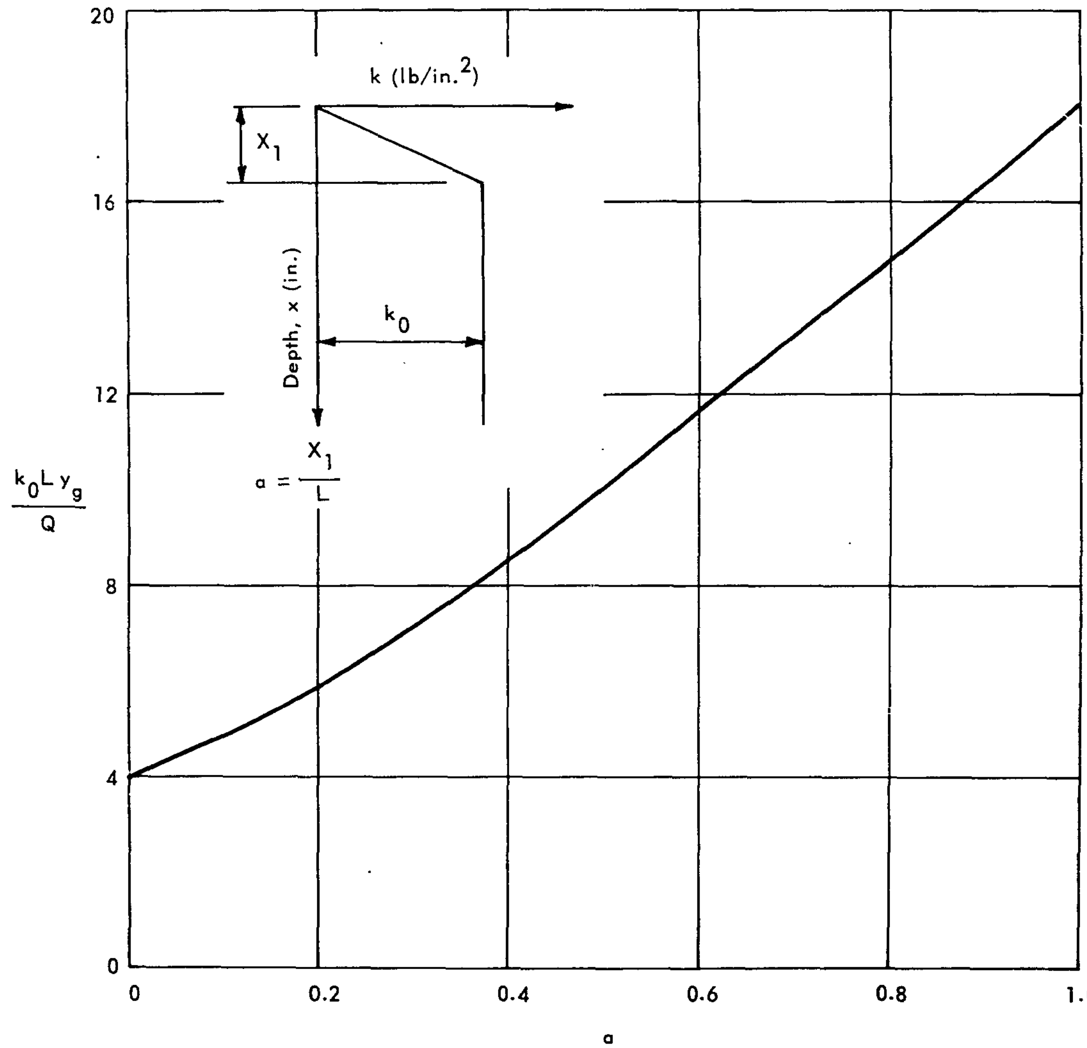

Figure 28. Pile-deflection parameter, layered granular soil.

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Key Words: rigid piles; lateral plates; moist beach sand

1. Structures — Rigid pile and lateral plates
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